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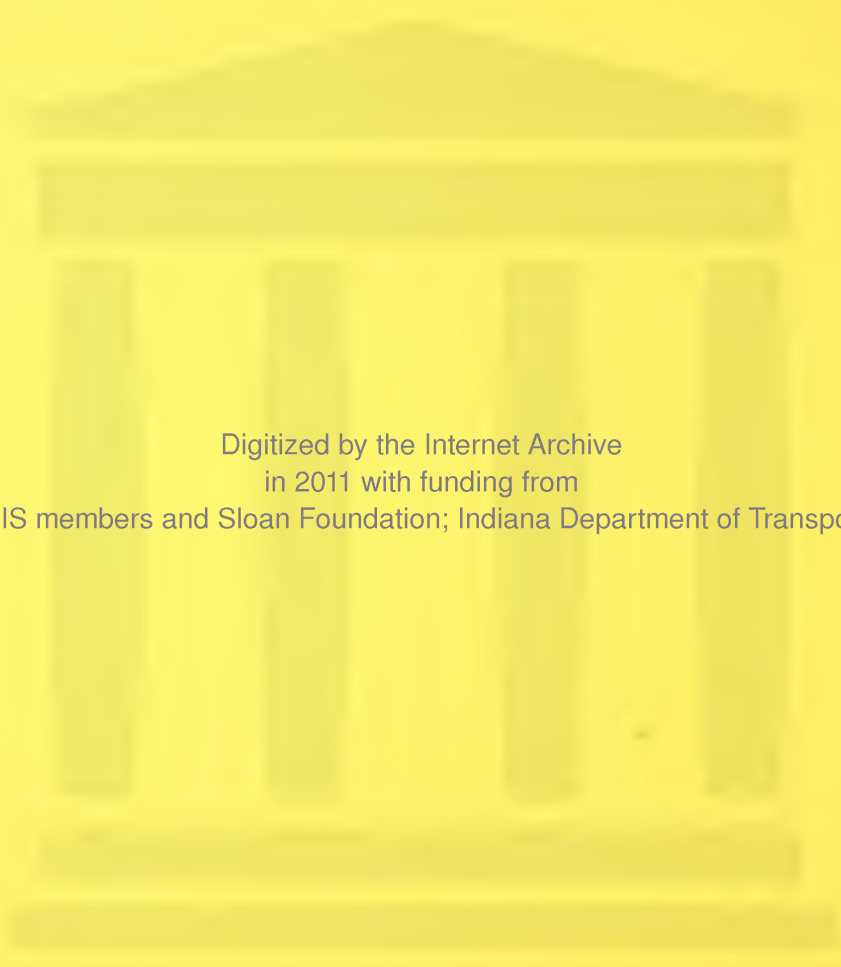
THE EFFECT OF LABORATORY
COMPACTION ON THE
SHEAR BEHAVIOR OF A HIGHLY
PLASTIC CLAY AFTER SATURATION
AND CONSOLIDATION

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PURDUE UNIVERSITY
INDIANA STATE HIGHWAY COMMISSION



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Interim Report

THE EFFECT OF LABORATORY COMPACTION ON THE SHEAR BEHAVIOR
OF A HIGHLY PLASTIC CLAY AFTER SATURATION AND CONSOLIDATION

TO: H. L. Michael, Director
Joint Highway Research Project

July 11, 1979

Project: C-36-5M

FROM: C. W. Lovell, Research Engineer
Joint Highway Research Project

File: 6-6-13

Attached is an Interim Report on the HPR Part II Study titled "Improving Embankment Design and Performance". This is Interim Report No. 6 and is titled "The Effect of Laboratory Compaction on the Shear Behavior of a Highly Plastic Clay After Saturation and Consolidation". It is authored by J. M. Johnson and C. W. Lovell of our staff.

The report describes the experimental program on the effective stress strength behavior of laboratory compacted St. Croix clay. The effects of compaction water content and effort level were evaluated for the clay after it had been saturated and consolidated under confining pressures simulating various embankment positions. Prediction equations were developed for the effective stress parameters, for the pore pressure parameter at failure, and for the volumetric strain during saturation and consolidation. These equations are largely in terms of the compaction variables. Research on these relationships for field compacted soil continues.

The Report is submitted for review, comment and acceptance as partial fulfillment of the objectives of the approved study.

Respectfully submitted,

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Interim Report

THE EFFECT OF LABORATORY COMPACTION ON THE SHEAR BEHAVIOR
OF A HIGHLY PLASTIC CLAY AFTER SATURATION AND CONSOLIDATION

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and the

U.S. Department of Transportation
Federal Highway Administration

The contents of this report reflect the views of the author who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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16. Abstract The effective stress strength behavior of the plastic St. Croix clay is to be studied for both laboratory and field compaction. This interim report deals with the laboratory-compacted phase. Shearing parameters were evaluated in the (CU) triaxial test after saturation and consolidation under confinements simulating a range of embankment positions. The friction angle, ϕ' , was found to be essentially invariant for the range of compaction conditions. The shearing intercept, c' , and the pore pressure parameter at failure, A_f , depended upon the final void ratio achieved for a selected consolidation pressure. Statistical analysis were performed to construct prediction equations for c' , A_f , and the volumetric strain on saturation and consolidation. These equations will aid efforts to predict and control the long term shear behavior of compacted clay embankments.			
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TABLE OF CONTENTS

LIST OF TABLES	Page vi
LIST OF FIGURES	ix
LIST OF ABBREVIATIONS AND SYMBOLS	xiv
HIGHLIGHT SUMMARY	xvii
INTRODUCTION	1
1 - LITERATURE REVIEW	4
1-1 Saturation of Compacted Soil	4
1-1.1 Introduction	4
1-1.2 Saturation by Percolation	5
1-1.3 Back Pressure Saturation	6
1-1.4 Methods for Checking Degree of Saturation	10
1-2 Strain Rates for Triaxial Testing	14
1-2.1 Effects of Strain Rate	14
1-2.2 Drained Triaxial Testing	16
1-2.3 Undrained Triaxial Testing	16
1-3 Fabric of Compacted Fine Grained Soils	21
1-3.1 Introduction	21
1-3.2 Particle Level Model	23
1-3.3 Aggregate Level Model	25
1-3.4 Changes During Swell and Consolidation	28
1-4 Long Term Behavior of Compacted Fine Grained Soils	30
1-4.1 Volume and Moisture Content Modifications	30
1-4.2 Effective Stress Strength Parameters	32
1-4.2.1 Drained Versus Undrained Testing	32
1-4.2.2 Variation of c' and ϕ' with Compaction Conditions	35
1-4.3 Stress-Strain Behavior	40
1-4.4 Pore Pressure Response During Undrained Shear	41
1-4.5 Consolidation to Anisotropic Versus Isotropic States of Stress	45

	Page
2 - APPARATUS AND PROCEDURE	49
2-1 Soil Processing and Classification	49
2-2 Moisture Addition and Curing	51
2-3 Compaction	53
2-3.1 Apparatus	53
2-3.2 Procedure	56
2-4 Sampling	60
2-4.1 High and Intermediate Sample Densities	60
2-4.2 Low Sample Densities	62
2-5 Sample Set-up	64
2-6 Back Pressure Saturation	67
2-7 Consolidation	71
2-8 Undrained Shear	72
2-9 Final Void Ratio Determination	74
2-10 Summary Listing of Independent Variables . .	75
3 - RESULTS AND DISCUSSION OF RESULTS	77
3-1 Compaction	77
3-1.1 Dry Density Versus Moisture Content Relationships	77
3-1.2 Definition of Compaction Variables . .	80
3-1.3 Sample Nomenclature	83
3-2 Saturation	84
3-3 Volume Change Due to Saturation and Consolidation	94
3-4 Stress-Strain and Pore Water Pressure Response	103
3-5 Effective Stress Strength Parameters	119
3-6 Statistical Analysis	129
3-6.1 Linear Regression Analysis	129
3-6.2 Dependent and Independent Variables .	133
3-6.3 Prediction Model for Volume Change due to Saturation and Consolidation .	141
3-6.4 Prediction Model for Skempton's A Parameter at Failure	152
3-6.5 Prediction Model for the Effective Stress Friction Angle, ϕ'	163
3-6.6 Prediction Model for the Effective Stress Strength Intercept, c'	164
3-7 Comparison of Strength Behavior of Samples Consolidated to Anisotropic and Isotropic States of Stress	169
4 - CONCLUSIONS AND RECOMMENDATIONS	176
4-1 Conclusions	176
4-2 Recommendations for Future Research	180

BIBLIOGRAPHY.	Page 182
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APPENDICES

Appendix A - Impact Compaction Data	192
Appendix B - Compaction, Saturation and Consolidation Data	194
Appendix C - Undrained Shear Data	198
Appendix D - Supplementary Figures.	253
Appendix E - Purdue University Negative Numbers for Photographs.	270

LIST OF TABLES

Table		Page
1.	EFFECTIVE STRESS STRENGTH PARAMETERS OF COMPACTED SOILS	37
2.	CLASSIFICATION TEST RESULTS FOR SAINT CROIX CLAY.	51
3.	SPECIFICATIONS FOR IMPACT COMPACTION TESTS. . . .	73
4.	BACK PRESSURES REQUIRED TO ATTAIN THE EXPERIMENTALLY OBSERVED DEGREES OF SATURATION . .	85
5.	VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION	100
6.	FAILURE CONDITIONS FOR CONSOLIDATED UNDRAINED TRIAXIAL TESTS.	114
7.	MEASURED VALUES OF THE EFFECTIVE STRESS STRENGTH PARAMETERS.	126
8.	BASIC COMPACTION VARIABLES FOR PERCENT VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION. . . .	134
9.	BASIC COMPACTION VARIABLES FOR SKEMPTON'S A PARAMETER AT FAILURE.	137
10.	BASIC COMPACTION VARIABLES FOR THE EFFECTIVE STRESS STRENGTH PARAMETERS.	139
11.	STATISTICAL DATA OF THE PREDICTION EQUATION FOR PERCENT VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION, $\Delta V/V_o(\%)$	142
12.	STATISTICAL DATA OF THE PREDICTION EQUATION FOR SKEMPTON'S A PARAMETER AT FAILURE, A_f	153
13.	STATISTICAL DATA OF THE PREDICTION EQUATION FOR THE EFFECTIVE STRESS STRENGTH INTERCEPT, c' . . .	165
14.	FAILURE CONDITIONS FOR ANISOTROPICALLY CON- SOLIDATED SAMPLES	174

Appendix

Table	Page
A1. MOISTURE CONTENT AND DRY DENSITY DATA FOR THE THREE IMPACT COMPACTION ENERGY LEVELS. . . .	193
B1. COMPACTION, SATURATION AND CONSOLIDATION DATA FOR ISOTROPICALLY CONSOLIDATED SAMPLES (CIU TESTS).	195
B2. COMPACTION, SATURATION AND CONSOLIDATION DATA FOR ANISOTROPICALLY CONSOLIDATED SAMPLES (CAU TESTS).	197
C1. UNDRAINED SHEARING RESPONSE OF SAMPLE MD1. . . .	199
C2. UNDRAINED SHEARING RESPONSE OF SAMPLE MD2. . . .	200
C3. UNDRAINED SHEARING RESPONSE OF SAMPLE MD3. . . .	201
C4. UNDRAINED SHEARING RESPONSE OF SAMPLE MD4. . . .	202
C5. UNDRAINED SHEARING RESPONSE OF SAMPLE MO1. . . .	203
C6. UNDRAINED SHEARING RESPONSE OF SAMPLE MO2. . . .	204
C7. UNDRAINED SHEARING RESPONSE OF SAMPLE MO3. . . .	205
C8. UNDRAINED SHEARING RESPONSE OF SAMPLE MO4. . . .	206
C9. UNDRAINED SHEARING RESPONSE OF SAMPLE MW1. . . .	207
C10. UNDRAINED SHEARING RESPONSE OF SAMPLE MW2. . . .	208
C11. UNDRAINED SHEARING RESPONSE OF SAMPLE MW3. . . .	210
C12. UNDRAINED SHEARING RESPONSE OF SAMPLE MW4. . . .	212
C13. UNDRAINED SHEARING RESPONSE OF SAMPLE MW5. . . .	214
C14. UNDRAINED SHEARING RESPONSE OF SAMPLE MW6. . . .	215
C15. UNDRAINED SHEARING RESPONSE OF SAMPLE SD1. . . .	216
C16. UNDRAINED SHEARING RESPONSE OF SAMPLE SD2. . . .	218
C17. UNDRAINED SHEARING RESPONSE OF SAMPLE SD3. . . .	220
C18. UNDRAINED SHEARING RESPONSE OF SAMPLE SD4. . . .	221
C19. UNDRAINED SHEARING RESPONSE OF SAMPLE SD5. . . .	222
C20. UNDRAINED SHEARING RESPONSE OF SAMPLE SD6. . . .	224

Appendix
Table

	Page
C21. UNDRAINED SHEARING RESPONSE OF SAMPLE SD7. . . .	226
C22. UNDRAINED SHEARING RESPONSE OF SAMPLE SO1. . . .	228
C23. UNDRAINED SHEARING RESPONSE OF SAMPLE SO2. . . .	230
C24. UNDRAINED SHEARING RESPONSE OF SAMPLE SO3. . . .	232
C25. UNDRAINED SHEARING RESPONSE OF SAMPLE SO4. . . .	234
C26. UNDRAINED SHEARING RESPONSE OF SAMPLE SW1. . . .	236
C27. UNDRAINED SHEARING RESPONSE OF SAMPLE SW2. . . .	237
C28. UNDRAINED SHEARING RESPONSE OF SAMPLE SW3. . . .	238
C29. UNDRAINED SHEARING RESPONSE OF SAMPLE SW4. . . .	239
C30. UNDRAINED SHEARING RESPONSE OF SAMPLE LD1. . . .	240
C31. UNDRAINED SHEARING RESPONSE OF SAMPLE LD2. . . .	241
C32. UNDRAINED SHEARING RESPONSE OF SAMPLE LD3. . . .	242
C33. UNDRAINED SHEARING RESPONSE OF SAMPLE LD4. . . .	243
C34. UNDRAINED SHEARING RESPONSE OF SAMPLE LO1. . . .	244
C35. UNDRAINED SHEARING RESPONSE OF SAMPLE LO2. . . .	245
C36. UNDRAINED SHEARING RESPONSE OF SAMPLE LO3. . . .	246
C37. UNDRAINED SHEARING RESPONSE OF SAMPLE LO4. . . .	247
C38. UNDRAINED SHEARING RESPONSE OF SAMPLE LO5. . . .	248
C39. UNDRAINED SHEARING RESPONSE OF SAMPLE LO6. . . .	249
C40. UNDRAINED SHEARING RESPONSE OF SAMPLE LW1. . . .	250
C41. UNDRAINED SHEARING RESPONSE OF SAMPLE LW2. . . .	251
C42. UNDRAINED SHEARING RESPONSE OF SAMPLE LW3. . . .	252

LIST OF FIGURES

Figure		Page
1.	BACK PRESSURE MAGNITUDE REQUIRED TO ATTAIN DESIRED SATURATION LEVEL (After Lowe and Johnson, 1960)	8
2.	B PARAMETER VS DEGREE OF SATURATION FOR SOILS OF VARYING STIFFNESS (After Black and Lee, 1973)	13
3.	TIME FACTORS FOR PORE PRESSURE EQUALIZATION DURING TESTING (After Blight, 1963)	22
4.	CHARACTERISTICS OF A SATURATED COHESIVE SOIL TESTED IN THE $\overline{CU}$ MODE (After Bishop and Bjerrum, 1960)	34
5.	CONTOURS OF PERCENT VOLUME CHANGE (After Abeyesekera, 1978)	38
6.	INFLUENCE OF SOIL FABRIC ON $\overline{CU}$ TEST RESULTS (After Seed and Chan, 1959)	39
7.	STRESS STRAIN CURVE CLASSIFICATION (After Casagrande and Hirschfeld, 1962)	42
8.	A_f VERSUS OVERCONSOLIDATION RATIO (After H�nkel, 1956)	44
9.	EFFECT OF K_0 CONSOLIDATION ON EFFECTIVE STRESS PATHS FOR $\overline{CU}$ TRIAXIAL TESTS ON NORMALLY CON- SOLIDATED SIMPLE CLAY (After Ladd, 1964)	47
10.	GRAIN SIZE DISTRIBUTION	50
11.	CALIFORNIA KNEADING COMPACTOR	54
12.	LOAD VERSUS TIME PLOTS FOR THE KNEADING COMPACTOR FOOT.	55
13.	DYNAMIC FOOT PRESSURE VERSUS GAGE PRESSURE, CALIFORNIA KNEADING COMPACTOR	57
14.	PROCTOR COMPACTION MOLD WITH KNEADING COMPACTOR MODIFICATIONS	58

Figure		Page
15.	SAMPLING METHOD FOR DENSE COMPACTION SPECIMENS.	61
16.	ATTACHMENTS FOR SAMPLE EXTRUSION	63
17.	WIRE SAW TRIMMING APPARATUS.	65
18.	DISASSEMBLED TRIAXIAL CELL	66
19.	SCHEMATIC OF THE SATURATION-CONSOLIDATION SYSTEM	68
20.	DRY DENSITY VERSUS MOISTURE CONTENT CURVES FROM IMPACT COMPACTION TESTS	79
21.	DRY DENSITY VERSUS MOISTURE CONTENT CURVES FROM KNEADING COMPACTION TESTS	81
22.	COMPARISON OF INITIAL AND FINAL VALUES OF DEGREE OF SATURATION	86
23.	COMPARISON OF INITIAL AND CORRECTED FINAL VALUES OF DEGREE OF SATURATION	90
24.	COMPARISON OF EXPERIMENTAL AND THEORETICAL BACK PRESSURES	92
25.	COMPARISON OF EXPERIMENTAL AND CORRECTED THEORETICAL BACK PRESSURES	93
26.	MOISTURE CONTENT AND DRY DENSITY VALUES BEFORE AND AFTER SATURATION AND CONSOLIDATION (COMPACTION DRY OF OPTIMUM MOISTURE CONTENT)	96
27.	MOISTURE CONTENT AND DRY DENSITY VALUES BEFORE AND AFTER SATURATION AND CONSOLIDATION (COMPACTION AT OPTIMUM MOISTURE CONTENT)	97
28.	MOISTURE CONTENT AND DRY DENSITY VALUES BEFORE AND AFTER SATURATION AND CONSOLIDATION (COMPACTION WET OF OPTIMUM MOISTURE CONTENT)	98
29.	CONTOURS OF PERCENT VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION.	99
30.	FINAL VOID RATIO VERSUS LOGARITHM OF CONSOLIDATION PRESSURE.	102
31.	$\overline{CIU}$ RESULTS, TESTS SD1 TO SD3.	104
32.	$\overline{CIU}$ RESULTS, TESTS SD4 TO SD7.	105

Figure		Page
33.	$\overline{CIU}$ RESULTS, TESTS SO1 TO SO4	106
34.	$\overline{CIU}$ RESULTS, TESTS SW1 TO SW4	107
35.	RELATIONSHIP BETWEEN DEVIATOR STRESS AT FAILURE AND FINAL VOID RATIO	110
36.	RELATIONSHIP BETWEEN PORE PRESSURE AT FAILURE AND FINAL VOID RATIO	111
37.	SKEMPTON'S A PARAMETER AT FAILURE VERSUS CONSOLIDATION PRESSURE	116
38.	SKEMPTON'S A PARAMETER AT FAILURE VERSUS AS-COMPACTED DRY DENSITY	117
39.	SKEMPTON'S A PARAMETER AT FAILURE VERSUS COM- PACTION MOISTURE CONTENT	118
40.	EFFECTIVE STRESS PATHS, TESTS SD1 TO SD3	120
41.	EFFECTIVE STRESS PATHS, TESTS SD4 TO SD7	121
42.	EFFECTIVE STRESS PATHS, TESTS SO1 TO SO4	122
43.	EFFECTIVE STRESS PATHS, TESTS SW1 TO SW4	123
44.	EFFECTIVE STRESS STRENGTH PARAMETERS AND ASSOCIATED COMPACTION CONDITIONS	125
45.	EFFECTIVE STRESS STRENGTH PARAMETER, c' , VERSUS FINAL VOID RATIO	128
46.	STANDARDIZED RESIDUALS VERSUS ESTIMATED PERCENT VOLUME CHANGE DUE TO SATURATION AND CONSOLI- DATION	143
47.	STANDARDIZED RESIDUALS VERSUS AN INDEPENDENT VARIABLE OF THE PREDICTION EQUATION FOR VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION . . .	144
48.	STANDARDIZED RESIDUALS VERSUS AN INDEPENDENT VARIABLE OF THE PREDICTION EQUATION FOR VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION . . .	145
49.	PREDICTION OF PERCENT VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION FOR AN INITIAL SATURATION OF 80%	146

Figure	Page
50. PREDICTION OF PERCENT VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION AT A CONSOLIDATION PRESSURE OF 170 kN/m^2	147
51. JOINT REGION OF INITIAL PERCENT SATURATION AND DRY DENSITY OBSERVATIONS	149
52. PREDICTION NOMOGRAPH FOR PERCENT VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION, $\Delta \hat{V}/V_o(\%)$	150
53. STANDARDIZED RESIDUALS VERSUS ESTIMATED VALUE OF SKEMPTON'S A PARAMETER AT FAILURE	154
54. STANDARDIZED RESIDUALS VERSUS AN INDEPENDENT VARIABLE OF THE PREDICTION EQUATION FOR SKEMPTON'S A PARAMETER AT FAILURE.	155
55. STANDARDIZED RESIDUALS VERSUS AN INDEPENDENT VARIABLE OF THE PREDICTION EQUATION FOR SKEMPTON'S A PARAMETER AT FAILURE.	156
56. PREDICTION OF SKEMPTON'S A PARAMETER AT FAILURE FOR AN INITIAL SATURATION OF 80%	157
57. EFFECT OF OVERCONSOLIDATION RATIO ON THE RELATIONSHIP BETWEEN FINAL VOID RATIO AND CONSOLIDATION PRESSURE	159
58. PREDICTION OF SKEMPTON'S A PARAMETER AT FAILURE FOR A CONSOLIDATION PRESSURE OF 170 kN/m^2	160
59. PREDICTION NOMOGRAPH FOR SKEMPTON'S A PARAMETER AT FAILURE, $\hat{A}_f$	162
60. STANDARDIZED RESIDUALS VERSUS THE INDEPENDENT VARIABLE OF THE PREDICTION EQUATION FOR THE EFFECTIVE STRESS STRENGTH INTERCEPT.	166
61. PREDICTION OF EFFECTIVE STRESS STRENGTH PARAMETER, c'	167
62. EFFECT OF MAXIMUM PAST PRESSURE OF THE EFFECTIVE STRESS STRENGTH INTERCEPT.	168
63. JOINT REGION OF INITIAL VOID RATIO AND MOISTURE CONTENT OBSERVATIONS	170
64. $\overline{CAU}$ RESULTS, TESTS LO7 TO LO10	172
65. EFFECTIVE STRESS PATHS, TESTS LO7 to LO10.	173

Appendix

Figure

	Page
D1. $\overline{\text{CIU}}$ RESULTS, TESTS MD1 TO MD4.	254
D2. $\overline{\text{CIU}}$ RESULTS, TESTS MO1 TO MO4.	255
D3. $\overline{\text{CIU}}$ RESULTS, TESTS MW1 TO MW3.	256
D4. $\overline{\text{CIU}}$ RESULTS, TESTS MW4 TO MW6.	257
D5. $\overline{\text{CIU}}$ RESULTS, TESTS LD1 TO LD4.	258
D6. $\overline{\text{CIU}}$ RESULTS, TESTS LO1 TO LO3.	259
D7. $\overline{\text{CIU}}$ RESULTS, TESTS LO4 TO LO6.	260
D8. $\overline{\text{CIU}}$ RESULTS, TESTS LW1 TO LW3.	261
D9. EFFECTIVE STRESS PATHS, TESTS MD1 TO MD4	262
D10. EFFECTIVE STRESS PATHS, TESTS MO1 TO MO4	263
D11. EFFECTIVE STRESS PATHS, TESTS MW1 TO MW3	264
D12. EFFECTIVE STRESS PATHS, TESTS MW4 TO MW6	265
D13. EFFECTIVE STRESS PATHS, TESTS LD1 TO LD4	266
D14. EFFECTIVE STRESS PATHS, TESTS LO1 TO LO3	267
D15. EFFECTIVE STRESS PATHS, TESTS LO4 TO LO6	268
D16. EFFECTIVE STRESS PATHS, TESTS LW1 TO LW3	269

LIST OF ABBREVIATIONS AND SYMBOLS

A	Skempton's A parameter
A_f	A parameter at failure
AASHTO	American Association of State Highway and Transportation Officials
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
b	Regression parameter
B	Skempton's B parameter
c'	Effective stress strength intercept
c_v	Coefficient of consolidation
$\overline{CU}$	Isotropically consolidated undrained triaxial shear test with pore pressure measurements
$\overline{CAU}$	Anisotropically consolidated undrained triaxial shear test with pore pressure measurements
$\overline{CU}$	Consolidated undrained triaxial shear test with pore pressure measurements
CD	Consolidated drained triaxial shear test
cm	Centimeter
e_o	Initial void ratio
e_f	Final void ratio
Δe	$(e_o - e_f)$
ft	feet
G_s	Specific gravity of soil solids
H	Henry's coefficient of solubility

in	Inch
K_o	Coefficient of earth pressure at rest
kg	Kilogram
kJ	KiloJoule
kN	KiloNewton
lb	Pound
log	Base 10 logarithm
m	Meter
mm	Millimeter
MSE	Mean Squared Error
n	Porosity
OCR	Overconsolidation ratio
p'	Mean effective normal stress = $(\sigma'_1 + \sigma'_3)/2$
p_c	Kneading compactor foot pressure
$\hat{p}_s$	Estimated compactive prestress
psi	Pounds per square inch
%	Percent
ϕ'	Effective stress friction angle
ρ_d	Dry density
ρ	Wet density
q	Shear stress = $(\sigma_1 - \sigma_3)/2$
R^2	Coefficient of multiple determination
R_a^2	Adjusted R^2
S_i	Initial degree of saturation (%)
S_f	Final degree of saturation (%)
σ	Total normal stress (kN/m^2)

σ'	Effective normal stress (kN/m^2)
σ'_1	Effective major principal stress (kN/m^2)
σ'_3	Effective minor principal stress (kN/m^2)
σ'_c	Consolidation pressure (isotropic) (kN/m^2)
$(\sigma_1 - \sigma_3)_f$	Deviator stress at failure (kN/m^2)
τ	Shear stress (kN/m^2) .
T	Time factor
t_f	Triaxial test duration
Δu_T	Theoretical back pressure
Δu_E	Experimental back pressure
Δu	Pore pressure change during shear
Δu_f	Pore pressure at failure
$\Delta V/V_o (\%)$	Percent volume change (volumetric strain) due to saturation and consolidation
w	Moisture content (%)
ϵ_a	Axial compressive strain

HIGHLIGHT SUMMARY

Compaction is a mechanical soil stabilization process which densifies the soil through expulsion of pore air at constant moisture content. The purpose of compaction is to increase the shear strength of the soil mass, reduce its compressibility and control its permeability and other related properties. Control of these engineering properties for the soil in its as-compacted state is commonly achieved through compaction specifications which place limits on the acceptable values of as-compacted dry density and compaction moisture content. However, the soil mass and its engineering properties are altered with time by environmental factors such as wetting and the load applied by succeeding lifts. To be certain of the satisfactory performance of compacted soil structures, the compaction specifications must account for volume changes over the long term and the attendant alteration of the shear behavior of the soil mass.

The effective stress strength parameters are utilized for analysis of long term stability. These parameters were evaluated for various compaction conditions through the

performance of consolidated undrained triaxial tests with pore water pressure measurement ($\overline{CU}$) at a constant rate of strain on kneading compacted samples of a highly plastic residual clay. The long term environmental effects were simulated with back pressure saturation and consolidation to an isotropic state of stress representing the load applied by succeeding lifts.

The results of the testing program showed that the effective stress friction angle is relatively invariant for the range of compaction conditions investigated. The magnitude and range of variation of the measured values of the effective stress strength intercept were small. However, the intercept values were observed to be a function of the final void ratio value attained for a selected consolidation pressure. This implies a dependency upon the percent volume change due to saturation and consolidation, which, in turn implies a dependency upon the swell potential of each compaction condition. The values of Skempton's A parameter at failure obtained in this study also indicated dependency on the final void ratio achieved for a selected consolidation pressure.

Statistical analyses were performed to construct prediction equations based on the compaction variables for percent volume change due to saturation and consolidation, Skempton's A parameter at failure, and the effective stress strength intercept. The equations obtained for percent

volume change due to saturation and consolidation and Skempton's A parameter at failure indicate that as-compacted dry density, initial degree of saturation and consolidation pressure are the critical variables influencing behavior. The equation for the effective stress strength intercept indicates that initial void ratio and compaction moisture content are the critical variables. These equations will aid efforts to predict and control the long term shear behavior of compacted cohesive soils.

INTRODUCTION

The extensive use of soil in highway embankments and earth dams is attributed to the satisfactory engineering properties which can be imparted to a soil mass through compaction. Loosely placed soil displays low shear strength, high compressibility and high permeability. Compaction densifies the soil through the expulsion of pore air at constant moisture content and produces greater shear strength, lower compressibility and lower permeability.

Commonly, field control of the engineering properties of a mass of compacted soil has been maintained through the use of compaction specifications which place a lower bound on the acceptable values of as-compacted dry density achieved for a limited range of compaction moisture contents. Such specifications have been utilized very successfully to control as-compacted (partially saturated) behavior. However, these specifications may not account for the possible alteration of soil properties with time induced by environmental factors. The compacted soil consolidates under the load applied by succeeding lifts and portions of the soil mass may approach saturation over the long term. The saturation process can induce large volume changes within the

embankment if the soil has significant swell potential or is susceptible to collapse.

The satisfactory performance of a compacted soil structure over the long term requires knowledge of the relationships among the compaction variables, which can be controlled during construction, and the engineering properties of the soil mass after loading and wetting. Scott (1977) and Price (1978) analyzed the unconfined compressive strength of a CL soil in its as-compacted condition and after soaking for both field and laboratory compaction. Such studies allow the engineer to predict the as-compacted and long term total stress strength of this compacted clay for very shallow depths of burial. Higher embankments require more sophisticated analysis, which takes into account compressibility and its effect on shear strength, in both the as-compacted and the long term cases. A number of studies were initiated to investigate these phenomena. DiBernardo (1979) studied the compressibility of laboratory compacted Saint Croix clay (CH) in its as-compacted condition and after saturation under load. The as-compacted shear behavior of Saint Croix clay under various confining pressures is currently under study by Weitzel (1979). The shear behavior of laboratory compacted Saint Croix clay after saturation and consolidation under various confining pressures was the subject of the research to be discussed in this report.

For the long term case, analysis of embankment stability is based on the effective stress strength parameters, c' and ϕ' . These parameters may be obtained from samples tested either in the consolidated drained mode (CD) or the consolidated undrained mode with pore water pressure measurements ($\overline{CU}$). The $\overline{CU}$ mode was selected for this study as it allows great savings in testing time. The samples for this testing program were formed in the laboratory with a kneading compactor and were later trimmed to the desired dimensions. Prior to shear they were back pressure saturated and consolidated under isotropic states of stress which represented the loads applied by succeeding lifts.

The ultimate purpose of this testing program was the construction of prediction equations for percent volume change due to saturation and consolidation ($\Delta V/V_o$, %), Skempton's A parameter at failure (A_f) and the effective stress strength parameters (c' and ϕ') based on the laboratory compaction variables. After completion of a similar study on field compacted samples, the prediction equations will be correlated with those obtained for the field compaction case. This correlation will provide the engineer with a valuable tool for controlling long term shear behavior.

1 - LITERATURE REVIEW

1-1 Saturation of Compacted Soil

1-1.1 Introduction

During compaction, soil is normally in a partially saturated state due to the inability of any compaction process to force all of the air from within the soil pore spaces. However, given an adequate external supply of water, long term conditions will include saturation or near saturation of portions of a compacted embankment. The time span required to attain this condition is a function of drainage provisions, soil permeability and the degree of attraction between soil and water (Abeyesekera, 1978).

Analysis of effective stresses is greatly facilitated by the use of saturated soil samples. The effective stress principle, shown to be valid by Bishop and Eldin (1950), is complicated by the presence of both water and air as pore fluids. Bishop (1960) discussed the laboratory procedures available for separate and simultaneous measurement of pore air and pore water pressures during triaxial testing. However, application of the measured pressures is made difficult by the existence of at least six different effective stress equations for partially saturated soils which

have been proposed during the last two decades (Fredlund and Morgenstern, 1977).

1-1.2 Saturation by Percolation

Prior to the widespread use and understanding of back pressure saturation, percolation of water through a compacted soil sample was the most commonly attempted method of saturation. Leonards (1955) reported the results of a procedure in which chamber pressure increments were accompanied by smaller pressure increments (20 to 30 percent) at one end of the sample. The opposite end of the sample was maintained at atmospheric pressure to allow air and water to escape. Saturation of a highly plastic compacted clay was termed impractical, while 97 percent saturation was consistently attained in a less plastic compacted loessial soil. Lowe and Johnson (1960) discussed the results of a procedure in which a high pressure at one end of the sample and a vacuum at the other were applied to induce flow. It was concluded that percolation can achieve high saturation levels for sands, but is unsatisfactory for cohesive soils. In addition, the high pressure gradients required to induce flow in unsaturated soils of low permeability imposed undesirable seepage forces on the samples.

The limited success of saturation by percolation is at least partially explained by the low permeability of partially saturated soil and the long flow path (full sample length). Yoshimi (1958) determined theoretically that the

water permeability of partially saturated soils decreases very rapidly as percent saturation decreases. Bjerrum and Huder (1957) determined experimentally that the permeability decreased with decreasing saturation level for samples at constant confining pressure, while Mitchell, Hooper and Campanella (1965) confirmed it for samples at constant volume. Matyas (1967) conducted permeability tests on two compacted fine grained soils and concluded that partial saturation is responsible for the "threshold gradient" phenomenon observed initially by Hansbo (1960).

1-1.3 Back Pressure Saturation

Application of a pressure increment to a pore fluid composed of free air and water compresses the air and with time drives some of it into solution. Boyle's Law governs the compressive behavior of the air, while Henry's Law of Solubility governs the process of air going into solution (Lowe and Johnson, 1960). Bishop and Eldin (1950) showed that the increase in pore pressure (Δu), necessary to saturate a soil sample decreasing in volume at constant water and total air content, is given theoretically by the following equation:

$$\Delta u = p_i (1 - S_i) / S_i H \quad (1-1)$$

p_i = initial pore air pressure

S_i = initial saturation level

H = Henry's coefficient of solubility

($H = 0.018 \text{ cm}^3 \text{ air per cm}^3 \text{ water at } 22^\circ\text{C}$)

The relationship is shown as a dashed curve in Figure 1 for an initial pore air pressure of one atmosphere. It should be noted that an initial pore air pressure greater than one atmosphere is possible for compaction moisture contents high enough to cause occlusion of the pore air. Lowe and Johnson (1960) presented the following theoretical relationship for the change in pore pressure (Δu) required to attain a desired level of saturation for a soil sample at constant volume and total air content and increasing water content.

$$\Delta u = p_i \frac{1 - S_i(1 - H)}{1 - S_f(1 - H)} - 1 \quad (1-2)$$

p_i = initial pore air pressure

S_i = initial saturation level

S_f = final saturation level

H = Henry's coefficient of solubility

This relationship is shown in Figure 1 as solid lines for an initial pore pressure of one atmosphere and desired saturation levels of 99 and 100 percent. The required pressures are lower than those calculated from Equation 1-1. From this trend it can be inferred that the pressure required to attain a desired level of saturation for a soil sample increasing in volume and water content with constant total air content is lower than that required in the previous two cases. It is therefore easiest to saturate samples which achieve net swell during the saturation and consolidation process.

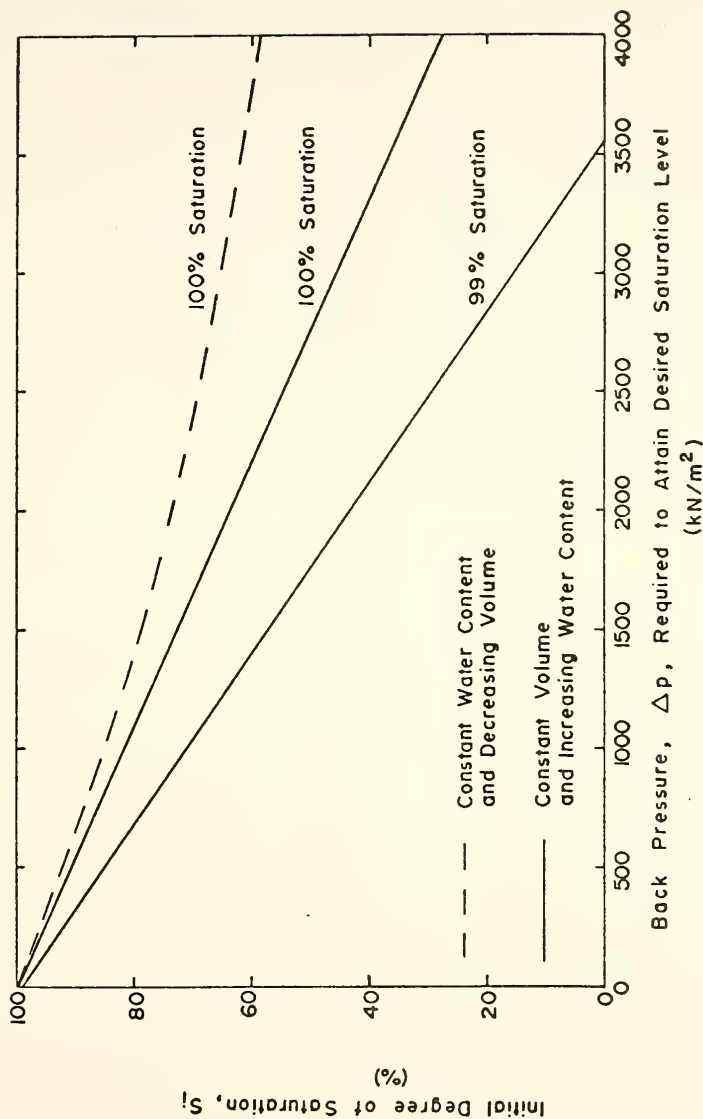


FIGURE 1 BACK PRESSURE MAGNITUDE REQUIRED TO ATTAIN DESIRED SATURATION LEVEL (After Lowe and Johnson, 1960)

The degree of saturation following application of a pressure increment to the pore fluid is a function of time as well as pressure (Black and Lee, 1973). The finite permeability of the soil impedes saturation, as the flow of sufficient water into the soil requires time. The process of compressed pore air going into solution is also a function of time. Bishop and Henkel (1962) and Blight (1963) reported that the use of longitudinal filter paper drains to shorten the minimum drainage distance greatly accelerated the flow of water for pore water pressure equilization during shear. Similarly, the flow for saturation is also accelerated.

Lowe and Johnson (1960) presented the general procedure for simultaneously increasing cell pressure and back pressure during the process of back pressure saturation. Bishop and Henkel (1962) reported for compacted soils that a series of increments of 35 to 70 kN/m^2 is commonly used, with final pressures ranging from 400 to 800 kN/m^2 . Of paramount importance during this procedure is the allowance of sufficient time for pore pressure equilization from one pressure increment to the next. If the pore fluid in the sample interior does not increase to the pressure level being maintained at the sample ends prior to application of the next pressure increment, the effective pressure in the sample interior is greater than that desired. If the time lag is great enough, the sample can become consolidated temporarily to a pressure higher than intended.

Back pressure saturation is a useful technique when testing soil samples which attempt to dilate during undrained shear. Cavitation in the pore pressure measuring system occurs when the pore pressure drops below 2.3 kN/m^2 absolute, the vapor pressure of water. Application of an initial back pressure to the sample allows measurement of changes in pore pressure which are negative while the actual pressure magnitudes remain positive.

1-1.4 Methods for Checking

Degree of Saturation

Three methods are used to determine the degree of saturation of test samples. Two of them can be applied during the saturation process, while the third is a post-test method. The first method, known as the B parameter check, is often applied just prior to each new pressure increment during back pressure saturation. Skempton (1954) defined the B parameter as the ratio of the change in pore pressure created by the undrained application of a hydrostatic pressure to the magnitude of that hydrostatic pressure. B can be determined at any time prior to failure by closing the drainage lines and measuring the pore pressure response to a small increment of chamber pressure. Further definition of the B parameter as a function of sample porosity and the relative compressibilities of the soil structure and pore fluid is provided by the following equation (Skempton, 1954).

$$B = \frac{\Delta u}{\Delta p} = \frac{1}{1 + n \frac{C_w}{C_s}} \quad (1-3)$$

Δp = hydrostatic pressure increment

Δu = undrained pore pressure response to Δp

n = porosity

C_w = compressibility of pore fluid

C_s = compressibility of soil skeleton

Wissa (1969) reported that the compressibility of the pore pressure measuring system requires additional terms in the equation, shown in equation (1-4), which are not negligible for very stiff soils.

$$B = \frac{\Delta u}{\Delta p} = \frac{1}{1 + n \frac{C_w}{C_s} + \left(\frac{V_L}{V_o}\right) \left(\frac{C'_w}{C_s}\right) + \frac{(C_L + C_m)}{C_s V_o}} \quad (1-4)$$

Δp , Δu , n , C_w and C_s = same as for equation (1-3)

C'_w = compressibility of fluid in pore water lines

C_L = compressibility of pore water lines

C_m = compressibility of pore pressure measuring element

V_L = volume of fluid in pore water lines

V_o = total volume of the test specimen

Although a B parameter of unity is often considered to represent complete saturation, these equations show that such a value may not be attainable for saturated soils with relatively stiff structures. Lee, Morrison and Haley (1969)

cited examples of saturated soils which produce B values considerably less than one. Shown in Figure 2 is a theoretical relationship developed by Black and Lee (1973) between B and the degree of saturation for various soil stiffness values. If a B of unity is unattainable, saturation can be considered complete when B becomes independent of the magnitude of back pressure.

The second method for checking degree of saturation can also be applied at any time prior to the application of axial load to failure. A back pressure is applied to the sample with the chamber fluid valve closed to prevent the sample from expanding. After subtracting the expansion of the pore pressure measuring system, the volume of flow into the sample is equal to the compression of the pore air (Abeyesekera, 1978). This method is often used inadvertently when monitoring water flow into the sample with the back pressure burette just prior to each cell and back pressure increment. If the sample is saturated and is not swelling, the only changes in water level will be due to system compressibility and evaporation at the air-water interface.

The third method is a post-test check of degree of saturation through the determination of the specific gravity of the soil solids and sample moisture content and volume. Sample volume is typically determined by applying Archimedes' principle after weighing the wax coated sample

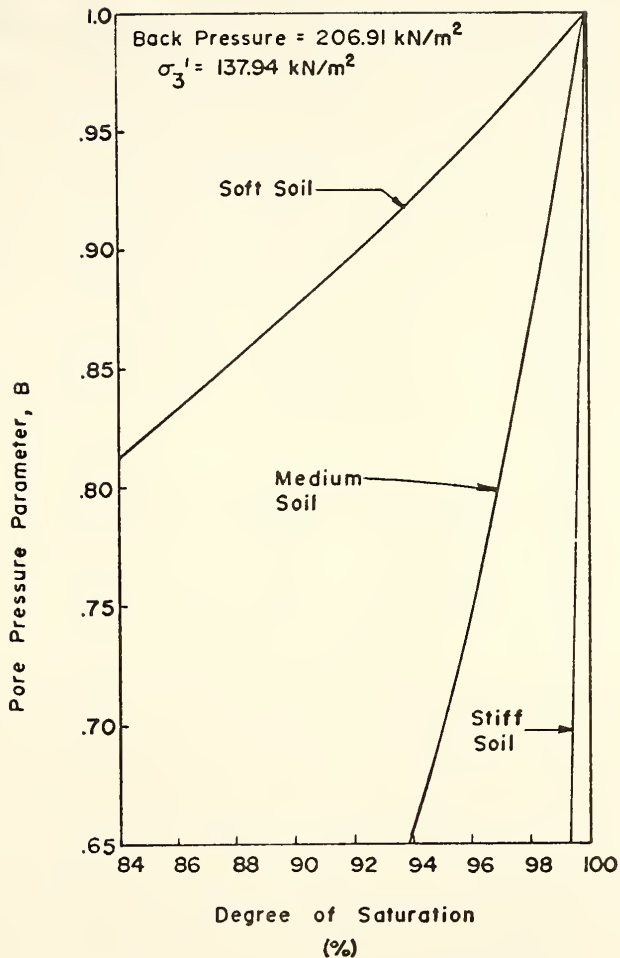


FIGURE 2 B PARAMETER VS DEGREE OF SATURATION
 FOR SOILS OF VARYING STIFFNESS
 (After Black and Lee, 1973)

in air and in water (Leonards, 1955 and Scott, 1977). A similar procedure is specified in AASHTO designation T233-70 for chunk density determination in the field. Menkel and Sowa (1963) reported that this method results in a higher water content than actually exists at failure. The negative pore water pressures induced upon release of the confining pressure can induce cavitation in the pore pressure measuring system and allow water to flow into the sample. However, Olson (1963c) reported that the increase in moisture content for soils of low permeability is limited to approximately 0.2 percent over the whole sample, and is therefore negligible for most purposes.

1-2 Strain Rates for Triaxial Testing

1-2.1 Effects of Strain Rate on Soil Behavior

The range of strain rate values commonly encountered in triaxial testing is bracketed by the inertial effects of extremely rapid loading and the creep effects of very slow loading. Within this range, compressive strength, induced pore pressure and void ratio changes are functions of strain rate. Casagrande and Wilson (1951) conducted undrained stress controlled tests on clays and clay-shales to show that saturated soils lose strength as the duration of each equal load increment is increased. Partially saturated soils were observed to initially lose and then gain strength

as duration was increased. Positive excess pore pressures, generated during undrained creep (Mitchell, 1976), may be responsible for the loss of strength reported for saturated soils. The strength gain for partially saturated soils was attributed by Casagrande and Wilson (1951) to a reduction in void ratio created by the compression and diffusion of pore air. Seed, Mitchell and Chan (1960) attributed the increase in part to thixotropy.

Undrained strain control tests performed on a saturated, highly plastic clay by Richardson and Whitman (1963) indicated that a decrease in strain rate reduces the ultimate compressive strength and increases the magnitude of the pore pressure change. Strain rates ranged from 1 percent per minute to 0.002 percent per minute. Whitman and Healy (1962) reported similar results from tests performed on saturated sands for times to failure ranging from 0.025 seconds to 3 minutes. Olson (1963a) also confirmed this trend, as long as failure is defined as the peak principal stress ratio.

Whitman (1966) stated that the effective stress strength parameters, c' and ϕ' , are not functions of strain rate. The observed variations in shear stress at failure can be attributed almost entirely to the concomitant variations in pore pressure. These statements are confirmed for saturated soils by the data of Whitman and Healy (1962) and Richardson and Whitman (1963). However, it was

emphasized that the pore pressure values must be reliable to observe this. The practical limitations imposed by the available testing equipment and procedures can produce misleading results.

1-2.2 Drained Triaxial Testing

Analysis of the results of drained triaxial tests is based upon the assumption that the excess pore pressures are fully dissipated. Gibson and Henkel (1954) reported that this assumption is continually violated, as complete dissipation is impractical for laboratory tests at constant rates of strain. It was shown for normally consolidated soils that the undissipated percentage of the pore pressure results in an underestimate of the angle of friction of the soil. To minimize the error, a theoretical relationship was derived which relates time to failure for the desired level of pore pressure dissipation to sample dimensions, drainage conditions and the coefficient of consolidation. Virtually all of the drained strength was observed to be attained if the chosen strain rate was that which theoretically allowed for 95 percent dissipation of the excess pore pressures.

1-2.3 Undrained Triaxial Testing

When the effective stress strength parameters are desired, analysis of the results of undrained triaxial tests is based on the assumption that the pore pressures have been reliably measured. However, Bishop, Blight and Donald (1960) reported that measured pore pressures are a function of

strain rate. This effect is caused by time lag in the pore pressure measuring system, modified behavior of the soil fabric and dissipation of the pore pressure gradients produced by nonhomogeneity of stresses and strains. With the exception of measurements made at very high strain rates, the time lag problem is not highly significant for truly saturated systems of low volume and compressibility (Bishop and Henkel, 1962). The modified behavior of the soil fabric is not critical as variations in pore pressure at failure are reflected in the compressive strength as discussed in subsection 1-2.1. However, the pore pressure gradients are a critical problem since theoretically an infinite number of pressures may be measured throughout the sample volume at a given instant. Variations in strain rate produce illusory variations in the effective stress strength parameters if only base and top measurements of pore pressure are made (Chan and Rivard, 1963 and Bishop, Blight and Donald, 1960).

Pore pressure gradients during shear result from nonhomogeneities of stress and strain within the soil sample which are produced by localized zones of weakness and restraint of the sample ends by the loading platens. Inclined zones of weakness, often a critical problem in natural soil deposits, are rarely a problem in compacted soils as the lift interfaces are horizontal or nearly so. End restraint creates zones of small shear strains at the sample ends

while a zone of large shear strains encompasses the remainder of the sample (Casagrande and Poulos, 1964). Perloff and Pombo (1969) conducted a theoretical investigation of stresses and strains in homogeneous right circular cylinders subjected to end restraint during loading. Their results indicated significant nonhomogeneity of stresses and strains as well as varying rotation of the planes of principal stress due to the development of shear stresses along the sample ends.

The existence of pore pressure gradients within a sample may be shown by direct measurement of pore pressures at various locations or by the indirect method of determining the final moisture content distribution. Whitman, Ladd and daCruz (1960) and Olson (1960) determined from moisture migrations that the direction of flow during shear is a function of overconsolidation ratio. Moisture migrated to the ends of normally consolidated soil samples, while heavily overconsolidated soil samples evidenced flow to the central portion. Bishop, Blight and Donald (1960) observed the same trend and confirmed the existence of the appropriate gradients by separate measurement of pore water pressures at the sample ends and center. Data from Richardson and Whitman (1963), Bishop, Alpan, Blight and Donald (1960) and Whitman (1960) indicate that the gradients are steepened by increase in strain rate.

There are three alternative approaches to the measurement of pore pressures which can reduce significantly the proportion of tests producing misleading results. The first involves the use of lubricated loading platens to reduce end restraint and allow the development of homogeneous stresses and strains. Rowe and Barden (1964) reported that the use of enlarged, lubricated platens allowed the use of shorter samples and reduced lateral piston loads, in addition to minimizing gradient formation within the samples. Measurement of pore pressure could be undertaken at any point within the sample without excessive error. However, the low friction values observed between lubricated platen and sample may not exist for all soil types and test durations. Casagrande and Poulos (1964) indicated that the equivalent friction angle of lubricated ends increased with decreasing soil stiffness and increasing duration of test. It is hypothesized that a time dependent and possibly local squeezing of lubrication grease from under the soil sample is responsible.

The second approach involves instrumenting the anticipated failure zone with a small pore pressure probe. The pore pressure measured is very likely to be closely related to the total stresses causing failure. Olson (1963d) cautioned that only very careful insertion of the pressure needles is adequate, as local disturbance of the sample and leakage of cell water between needle and membrane will alter

pore pressure response. Response is also affected by the stress-strain incompatibility of the soil and the implanted measuring devices.

The third approach involves the use of a strain rate which allows any induced pore pressure gradients to dissipate. The pore pressure measured at the sample ends is then essentially identical to that acting throughout the sample. Bishop (1961) reported that this method is very satisfactory for the determination of c' and ϕ' , but the values of pore pressure (and therefore Skempton's A parameter) will differ from those obtained by direct measurement and faster straining. It should also be noted that the moisture content in the failure zone at failure will possibly differ greatly from the original value, as gradient dissipation is accomplished by moisture migration.

Blight (1963) proposed a procedure for calculating test duration (t_f) required for adequate pore pressure gradient dissipation. As shown in the following equation, test duration is a function of desired percentage of dissipation, drainage conditions, sample dimensions and the coefficient of consolidation.

$$t_f = \frac{T H^2}{c_v} \quad (1-5)$$

T = time factor

H = 0.5 x specimen height for a 2:1 aspect ratio

c_v = coefficient of consolidation

The time factor, T , is obtained from the appropriate theoretical or experimental curve as a function of the desired percentage of dissipation. Theoretical curves representing various drainage conditions are shown in Figure 3. Bishop, Alpan, Blight and Donald (1960) reported that during the theoretical derivation of the curves it was assumed that the coefficient of consolidation is identical for consolidation and swelling, the nonhomogeneity of pore pressure is proportional to the stress increment and parabolic in shape, and the loading rate is constant. Blight (1963) concluded that 95 percent equalization is highly satisfactory for samples with overconsolidation ratios less than twenty. More highly overconsolidated samples display such steep pressure gradients that more than 95 percent equalization may be required to achieve acceptable error levels.

1-3 Fabric of Compacted

Fine Grained Soils

1-3.1 Introduction

Mitchell (1976) defines the terms fabric and structure for use with soils as follows:

"The term 'fabric' refers to the arrangement of particles, particle groups and pore spaces in a soil. The term 'structure' is used by some interchangeably with the term 'fabric'. Herein, however, the term 'structure' is taken to have the broader meaning of the combined effects of fabric, composition and interparticle forces."

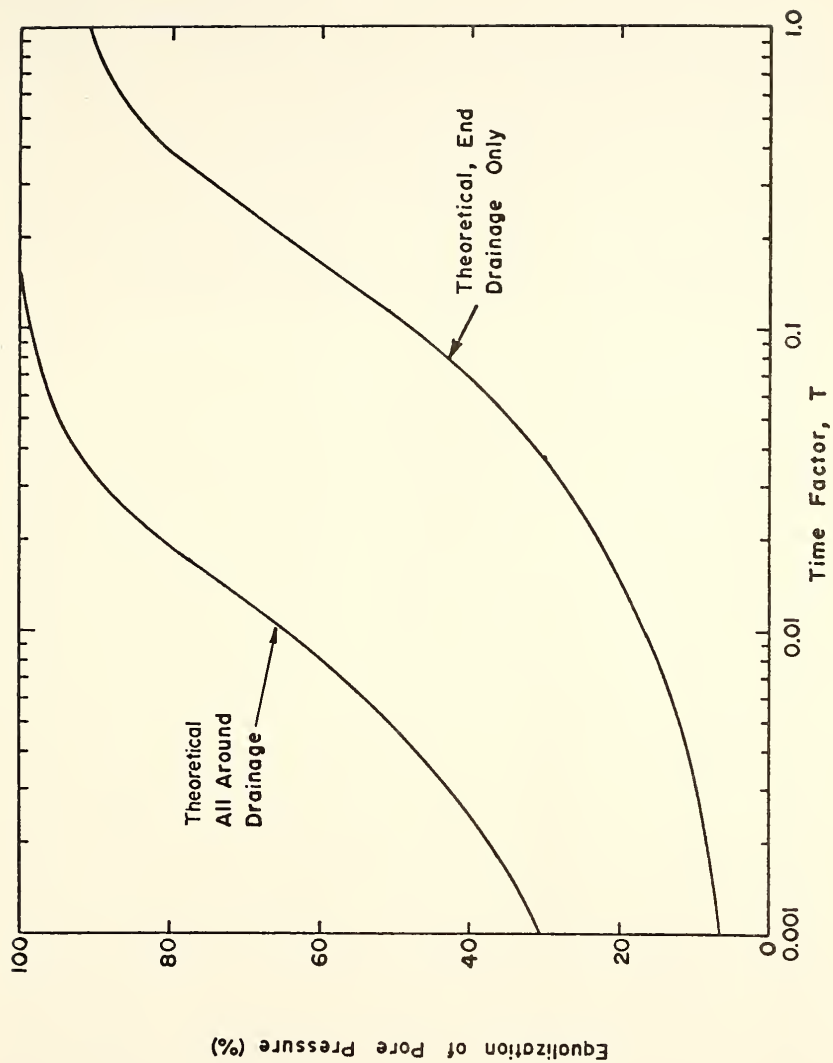


FIGURE 3 TIME FACTORS FOR PORE PRESSURE EQUALIZATION DURING TESTING
(After Blight, 1963)

He further defines fabric by dividing it into two levels, macrofabric and microfabric. Macrofabric, including voids, root holes, cracks, slickensides, stratification and compaction lift laminations, is generally more critical in terms of shear strength for natural than compacted soils and will not be considered further. Microfabric considers the behavior and orientation of individual soil particles or particle clusters and has been the focus of considerable study.

1-3.2 Particle Level Model

Proctor (1933) and Hogentogler (1936) developed theories for the mechanics of compaction which were based on the interaction between the soil water and the individual soil particles. Lambe (1958) developed this concept further with the aid of colloidal and crystal chemistry and embodied it in a model for the structure of compacted clay. The model is based on the concept of "water deficiency" which recognizes that there is rarely enough water in a compacted clay to give each individual soil particle all the water it could attract and hold. The attracted water is termed double layer water, whereas any unattracted excess is termed free water. At dry of optimum moisture contents, insufficient water and high electrolyte concentrations depress the double layer. The depression reduces interparticle repulsion and produces a flocculated fabric. As the optimum moisture content is approached, through the addition of

moisture to the soil, the double layer expands and the electrolyte concentration decreases. Lambe borrowed the term "lubrication" from the early concepts of Proctor and Hogentogler to describe the increased interparticle repulsion and lowered frictive forces which result in a denser, more orderly particle arrangement. Addition of moisture in excess of the optimum moisture content causes further expansion of the double layer, increasing interparticle repulsion and creating a highly ordered local particle arrangement. The compacted mass is now less dense as water occupies much of the space into which the compactive forces are trying to push the particles.

Seed and Chan (1959) utilized Lambe's model to explain the results of triaxial tests on compacted clay samples. Two samples, one compacted dry of optimum and the other wet of optimum, were soaked under a common confining pressure until they achieved equilibrium at a common dry density and moisture content. For axial strains less than 10 percent the "wet" sample sustained lower deviator stresses and higher pore pressures than the "dry" sample. The authors attributed this to the more dispersed, less rigid structure of the "wet" sample. For strains greater than 10 percent the differences between "wet" and "dry" samples were small. The authors attributed this to particle dispersion in the failure zone in response to shear strain. At large strains the fabric of the "dry" sample is destroyed and tends toward

that of the "wet" sample. The dispersion by shear strain concept was extended to explain variations in stress-strain behavior noted for samples of identical dry density and moisture content compacted by different means. For samples compacted wet of optimum moisture content it was concluded that static compaction produces less shearing and results in a less dispersed fabric than kneading compaction. At dry of optimum moisture contents, the differences in interpreted fabric were very small.

1-3.3 Aggregate Level Model

Olsen (1962) hypothesized an aggregate fabric model for soil, to explain discrepancies between experimental permeabilities and those obtained from the Kozeny-Carman equation. In this model, each spherical soil aggregate is composed of a large number of soil particles. Soil pores are either of the large inter-aggregate type or the small intra-aggregate type. The trend of the variations in permeability in response to variations in void ratio is reproduced very well by calculations which assume that changes in void ratio are absorbed primarily by the large inter-aggregate pores while the small intra-aggregate pores are incapable of transmitting significant flow. However, Olsen conducted no microfabric studies to confirm his hypothesis. Garcia-Bengochea (1978) used mercury porosimetry to determine the magnitude and frequency of the large pore mode and concluded that permeability is largely a function of the

size of the large pore mode and the distribution of pore sizes about it.

Barden and Sides (1978) utilized scanning electron microscopy and a battery of soil behavior measurements to arrive at an aggregate model of compacted soil fabric. Prior to compaction the packets of soil particles, or domains, were clustered into aggregates of visible and varying proportions. After compaction a significant difference was observed at low magnifications between the fabrics of soil compacted wet and dry of optimum. After compaction dry of optimum, the distinct packets or domains were still visible with large pores intervening. After compaction wet of optimum the individual domains and pores were no longer visible. High magnification revealed no significant differences between the particle level fabrics of "wet and dry" samples. The authors postulated that moisture contents dry of optimum produce soil packets and aggregations with sufficient strength to resist distortion during compaction and leave large inter-aggregate pore spaces. Higher moisture contents weaken the packets and aggregations allowing them to deform under load and reduce the volume of the large pores. Air permeability measurements of zero at optimum moisture content indicate that the deformation advanced sufficiently to occlude the large pores. Moisture contents wet of optimum produce very weak packets and aggregations

which deform sufficiently under load to become indistinguishable. The added volume of moisture displaces soil solids and causes dry density to decrease.

Hodek (1972) arrived at an aggregate model of compacted soil structure after conducting a study independent of that of Barden and Sides. He verified his "deformable aggregate" model by first investigating the particle orientation and strength of kaolin aggregates of varying moisture content and size. He then investigated the compaction characteristics of the aggregates and the swell characteristics of the compacted samples. It was concluded that the aggregate model provided an explanation of engineering behavior superior to that provided by the particle level model proposed by Lambe (1958). Hodek also suggested that understanding of the behavior of soil aggregates of varying moisture content under load may be advanced by application of the effective stress theory of soil compaction proposed by Olson (1963b).

Studies of compacted kaolinite and illite, reported by Diamond (1971), provided additional evidence for the deformable aggregate model. For samples compacted dry of optimum, scanning electron microscopy showed distinct soil domains several microns in diameter with intervening voids of similar diameter. The domains were indistinguishable and the voids were closed for samples compacted wet of optimum.

Bhasin (1975) utilized mercury porosimetry to compare the pore size distributions obtained from compaction to a given dry density at two moisture contents. He concluded that compaction dry of optimum produces a greater fraction of large pore sizes than does wet side compaction. Data for compacted Grundite (a CL soil) show a significant volume of pores with diameters of 0.5 to 2 μm for dry side compaction which are not present wet of optimum. Wet side compaction produced a greater fraction of pores with diameters of 0.07 to 0.5 μm while the fraction of very small pores (less than 0.07 μm) was roughly equal for both compaction moisture contents. Sridharan, Altschaeffl and Diamond (1971) reported that the reduction in void ratio obtained by increasing compaction effort is absorbed primarily by the large pores. Essentially identical pore size distributions for samples compacted by different methods to the same dry density and moisture content were reported by Ahmed, Lovell and Diamond (1974).

1-3.4 Changes During Swell and Consolidation

Lambe (1958) postulated that the phenomenon of swell upon inundation was the result of the simultaneous expansion of the double layer of each particle in its drive to satisfy its "water deficiency". As noted by Ladd (1960) and Parcher and Liu (1965), the percent swell achieved is a function of the type of clay mineral, the initial compaction conditions



and the confining pressure. Although there is a paucity of published microfabric studies on changes during swelling, Hodek (1972) has postulated a mechanism based on his aggregate level model. He incorporated Lambe's concept of expanding double layers to explain the observed swell pressures upon inundation. However, the magnitude of the swell pressure is diminished by the rearrangement and plastic deformation of the wetted soil aggregates. Hodek based his explanation of observed variation in swell pressure with time on this mechanism.

For soil compacted dry of optimum and placed under a confining pressure, collapse upon inundation becomes a significant possibility (Mishu, 1963). Hilf (1975) and Abeyesekera (1978) explain collapse as a shear failure in response to lowered shear strength due to destruction of capillary forces. The softened aggregates deform and fill in a portion of the void space. The particle level fabric is likely to be more ordered after collapse, the degree being a function of the amount of aggregate degradation and deformation.

Consolidation of compacted soil which has swelled or collapsed also causes rearrangement and deformation of the soil aggregates. In contrast to compaction, however, the author believes that the volume change is likely to be absorbed by both the small and the large pores.



1-4 Long Term Behavior of Compacted Fine Grained Soils

1-4.1 Volume and Moisture Content

Modifications

Long term conditions within a compacted embankment are the product of an interaction between changes in volume under load and moisture content modifications. Given an adequate supply of water, saturation or near saturation will occur in portions of a compacted embankment. The modification in properties created by wetting prompted Bishop and Henkel (1962) to prescribe only those tests which allow softening at the appropriate stress levels for determination of long term behavior. As long term pore pressures will include equilibrium and perhaps excess of equilibrium components, stability analysis should be based on the effective stress strength parameters.

Consolidation, swell and collapse are the mechanisms by which volume changes occur within a compacted embankment. Under the weight of succeeding lifts, soil compacted at or wet of optimum moisture content will achieve a time dependent reduction in volume similar to that predicted by classical consolidation theory (Danielson, 1963, Yoshimi, 1958, and Barden, 1974). For soil compacted dry of optimum the same investigators reported that the volume change process involved the compression and expulsion of pore air followed by drained creep. Pore water was not expelled

until volumetric reduction under load resulted in degrees of saturation in excess of 90 percent.

Upon wetting, soil compacted at or wet of optimum will attempt to swell. The percent swell achieved is a function of soil mineralogy, compaction moisture content and dry density, and the confining pressure supplied by succeeding lifts (Ladd, 1960 and Parcher and Liu, 1965). Woodsum (1951) reported that swell reduces the prestress established by compaction and increases the compressibility of compacted clay.

For soil compacted sufficiently dry of optimum, wetting can induce either swelling or collapse. Mishu (1963) noted from oedometer tests that there exists a critical load above which wetting will cause collapse and below which wetting causes swell. Hilf (1975) and Abeyesekera (1978) explain the collapse phenomenon as a shear failure in response to strength loss induced by the destruction of capillary tension. If the applied shear stresses do not exceed the reduced strength, swelling will occur instead.

Laboratory simulation of long term conditions is achieved through saturation of the compacted sample (discussed in section 1-1) and consolidation under the expected confining pressure prior to shear. Mishu (1963) presented data which indicated that the sequence of saturation and consolidation can affect the equilibrium void ratio. This stress path dependency is likely to be small for most

compaction conditions but was determined to be significant for compaction conditions susceptible to collapse. It is most likely that compression is concluded insitu prior to complete saturation. However, the use of back pressure saturation has resulted in the common laboratory procedure for consolidated drained (CD) and consolidated undrained (CU) triaxial tests, of saturating at low confining pressures prior to consolidation at the higher final confining pressure. Saturation prior to consolidation allows the consolidation process to be monitored with the water level in the back pressure burette. For overconsolidated samples, saturation at a low confining pressure is likely to reduce the void ratio changes caused by the cyclic loading induced during incrementation of the confining and back pressures.

1-4.2 Effective Stress Strength

Parameters

1-4.2.1 Drained Versus Undrained Testing

The effective stress strength parameters, c' and ϕ' , are sometimes obtained from samples tested in the consolidated drained (CD) mode. Following consolidation to the desired confining pressure each sample is sheared at a strain rate which allows no buildup of excess pore pressure. Times to failure to assure adequate drainage range from 8 to 50 hours for some typical clays with the majority requiring at least 20 hours (Bishop and Henkel, 1962). To reduce the required time to failure, c' and ϕ' are often determined

from samples tested in the consolidated undrained mode with measurement of pore water pressures ($\bar{C}U$). Following consolidation to the desired confining pressure, the drainage lines are closed and pore water pressures are measured during shear. Typical compression and shear strength relationships for $\bar{C}U$ tests on a saturated cohesive soil are shown in Figure 4. Bishop and Henkel (1962) noted that $\bar{C}U$ tests with end pore water pressure measurement on most typical clays can be completed in a working day. This assumes that pore pressure equalization throughout the 76.2 mm by 38.1 mm (3 in by 1.5 in) samples is desired only at failure and that longitudinal filter paper drains are used. A method for the calculation of times to failure for adequate pore pressure equalization was presented by Blight (1963), and is discussed in section 1-2.

The use of either CD or $\bar{C}U$ tests to define c' and ϕ' is an example of the concept that Ladd (1964) presents as follows, for simple, saturated clays.

"For normally consolidated samples, or for overconsolidated samples with the same maximum past pressure σ'_{cm} (considering shear in compression and extension separately) there is a unique relationship between strength (maximum stress difference = $(\sigma'_1 - \sigma'_3)_{max}$) and effective stress at failure."

Bjerrum and Simons (1960) compared ϕ' values from CD and $\bar{C}U$ tests for 19 normally consolidated clays in either an undisturbed or remolded state. They concluded that the ϕ' values obtained from $\bar{C}U$ tests using the $(\sigma'_1/\sigma'_3)_{max}$ maximum

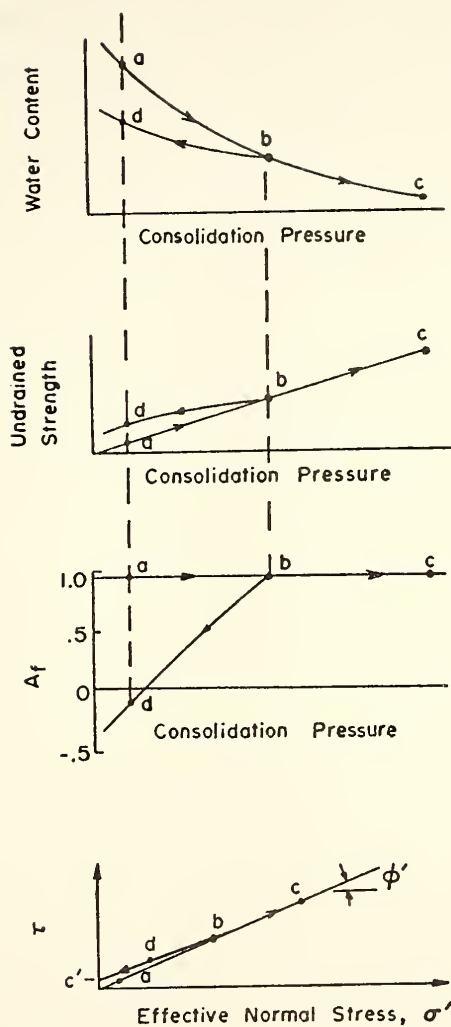


FIGURE 4 CHARACTERISTICS OF A SATURATED COHESIVE SOIL TESTED IN THE $\overline{CU}$ MODE. (After Bishop and Bjerrum, 1960)

failure criterion proposed by Holtz (1947) are in general agreement with the ϕ' values from CD tests. Values of ϕ' from $\overline{CU}$ tests using the $(\sigma_1 - \sigma_3)$ maximum failure criterion can be significantly lower. Bishop and Bjerrum (1960) reported for overconsolidated clays that ϕ' from CD testing is slightly larger than that obtained from $\overline{CU}$ tests at $(\sigma_1 - \sigma_3)$ maximum, as work is done during CD shear to increase the volume of the sample.

1-4.2.2 Variation of c' and ϕ' with Compaction Conditions

Knowledge of the relationships between the compaction conditions (dry density, moisture content and compaction energy) and the long term effective stress strength parameters would be a valuable engineering tool for controlling the long term shearing behavior. However, published data on the variation of the effective stress strength parameters with initial compaction conditions are limited in quantity and scope compared to that available for both short and long term total stress strength parameters.

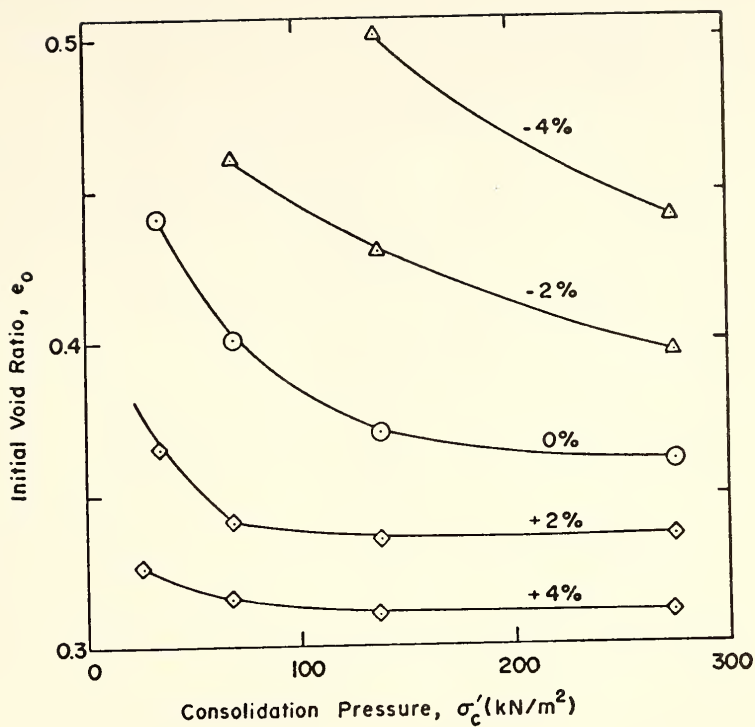
Casagrande and Hirschfeld (1960, 1962) presented the results of back pressure saturated, stress control $\overline{CU}$ triaxial testing on a clay of low plasticity. Effective stress strength parameters were obtained for various moisture contents at each of three dry density values. However, the data cannot be used with confidence as subsequent testing indicated that the times to failure utilized in the testing program were inadequate for pore water pressure equalization

throughout the samples. The pore pressures measured at the sample ends are believed to be significantly higher than those actually acting on the failure plane. Table 1 presents the results of other investigators. Linell and Shea (1960) reported c' and ϕ' values obtained from CD direct shear tests on four New England glacial tills compacted to standard Proctor densities. Table 1 shows that consistent trends are difficult to discern. Bazett and Bell (1963) presented the results of $\overline{CU}$ and CD testing on a glacial till compacted at two moisture contents and two dry densities. The limited number of compaction points makes trend interpretation difficult as can be seen in Table 1.

Abeyesekera (1978) reported for the limited range of compaction conditions investigated for a single test shale that the c' and ϕ' values did not vary significantly with these conditions. The key to this behavior is shown in Figure 5. Compacted samples with high initial void ratios displayed net volumetric compression after saturation and consolidation, while samples with low initial void ratios expanded. The variation in void ratio was therefore much smaller after saturation and consolidation than in the as-compacted condition. Seed and Chan (1959) presented shearing data, shown in Figure 6, for samples compacted dry and wet of optimum moisture content. The samples achieved identical dry densities and moisture contents after saturation under a common confining pressure. Although the shapes of the

TABLE 1. EFFECTIVE STRESS STRENGTH PARAMETERS OF COMPACTED SOILS

Investigators	Soil	Test Type	Compaction Conditions			Effective Stress Strength Intercept c' (kN/m ²)	Effective Stress Friction Angle, ϕ' (Degrees)
			Deviation From Standard Proctor Optimum Moisture Content (%)	Deviation From Standard Proctor Optimum Dry Density (kg/m ³)			
Linell and Shea (1960)	Otter Brook Till	CD (Direct Shear)	-1.0	--		0.0	33
			0.0	--		0.0	32
			+2.0	--		0.0	31
Linell and Shea (1960)	Hopkinton Till	CD (Direct Shear)	-2.1	--		0.0	32
			-0.2	--		0.0	39
			+1.7	--		0.0	35
Linell and Shea (1960)	Ball Mountain Till	CD (Direct Shear)					
Linell and Shea (1960)	Thomaston Dam Till	CD (Direct Shear)	-2.0	--		0.0	34
			0.0	--		9.6	33
			+2.0	--		19.2	35
Bazett and Bell (1963)	Little Long Rapids Till	CD and CU (Triaxial)	0.0	0.0		8.3	37
			-2.5	0.0		27.6	31
			-2.5	+96.1		20.7	37



- △ Contraction (-)
- Zero Volume Change
- ◇ Expansion (+)

FIGURE 5

CONTOURS OF PERCENT
VOLUME CHANGE
(After Abeyesekera, 1978)

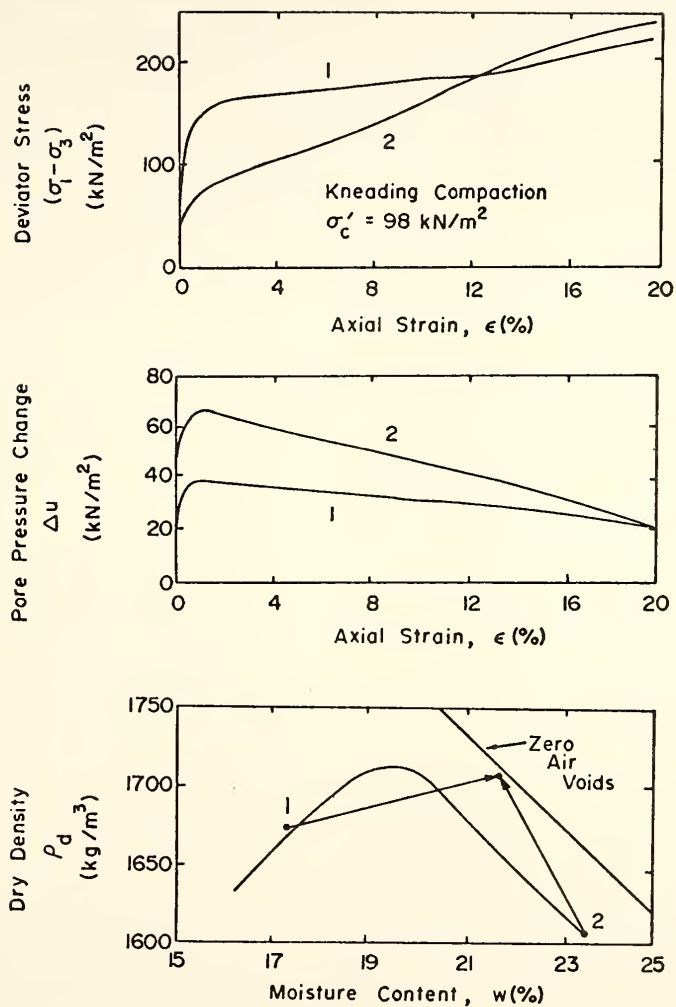


FIGURE 6 INFLUENCE OF SOIL FABRIC ON $\bar{C}_U$ TEST RESULTS (After Seed and Chan, 1959)

stress-strain and pore pressure change curves evinced the different soil fabrics created by the different compaction conditions, the shear strengths and pore pressure changes at failure were essentially equal for the two samples. Therefore, it is possible that the limited variations in c' and ϕ' observed by Abeysekera (1978) resulted from small differences in shear strength and failure pore pressures caused by limited variation in void ratio after saturation and consolidation.

1-4.3 Stress-Strain Behavior

Seed and Chan (1959) reported that the shape of the stress-strain curve for saturated compacted soils is a function of the soil fabric produced by the initial compaction conditions. They postulate a flocculated fabric for moisture contents dry of optimum and a dispersed fabric wet of optimum for kneading compaction. The original soil fabric is altered by swell during wetting, but not completely destroyed. Figure 6 compares the stress-strain behavior and pore pressure response of two samples, one compacted dry of optimum and one compacted wet of optimum, which were soaked and consolidated to a common dry density and moisture content prior to shear. The fabric postulated to be flocculated, displays more brittle behavior than that postulated to be dispersed.

Casagrande and Hirschfeld (1962) proposed a classification system for the shapes of stress-strain curves which is shown in Figure 7 and explained as follows:

- Type I : Displays a well-rounded shape with no initial straight line portion.
- Type II : A type I curve with the exception of an initial straight line portion up to a point short of 50 percent of the compressive strength.
- Type III: Displays a steep straight line portion which culminates in a yield point above 50 percent of the compressive strength. Resistance changes only slightly as straining continues.
- Type IV : A type III curve with the exception that resistance increases significantly following yield.
- Type V : A type IV curve with the exception that yield occurs at or less than 33 percent of the compressive strength.

The subdivisions within each type (a,b,c,) simply indicate whether resistance increases, stays constant or decreases following yield.

1-4.4 Pore Pressure Response

During Undrained Shear

The volume change tendencies of a saturated soil sample determine the changes in pore pressure which occur during undrained shear. Normally consolidated and lightly overconsolidated clays tend to compress when loaded and develop positive pore water pressures if sheared undrained.

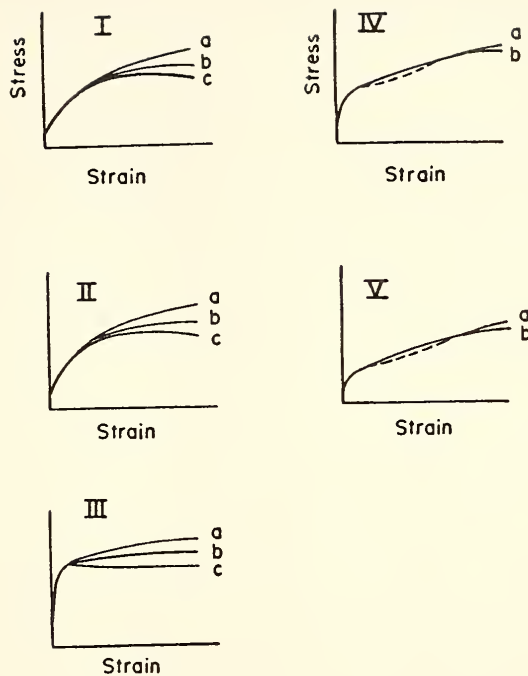


FIGURE 7 STRESS STRAIN CURVE
CLASSIFICATION
(After Casagrande and Hirschfeld, 1962)

Highly overconsolidated clays tend to dilate when loaded and develop negative pore water pressures if sheared undrained.

Skempton (1954) proposed the following equation for pore pressure response during triaxial compression.

$$\Delta u = B \{ \Delta \sigma_3 + A(\Delta \sigma_1 - \Delta \sigma_3) \}$$

"B", an empirical coefficient related to the degree of saturation and sample stiffness, is obtained as the ratio of the pore pressure change to the all around pressure increment which induced it. It is further discussed in section 1-1. "A" is an empirical coefficient which describes the pore pressure response to shear stress. Henkel (1965) presented data which showed that the magnitude of the A parameter at failure is a function of overconsolidation ratio for a given soil with a given stress history. Figure 8 shows the results obtained for remolded Weald and London clays. Abeyesekera (1978) reported a similar relationship between the A parameter at failure and the ratio of consolidation pressure to compaction pressure for a shale compacted at a given compaction pressure and moisture content. However, there is a dearth of information on the variation of the A parameter at failure with compaction conditions (dry density, moisture content and compactive energy) for a given consolidation pressure. Lambe and Whitman (1969) reported that A parameter values at failure can range from 2 to 3 for

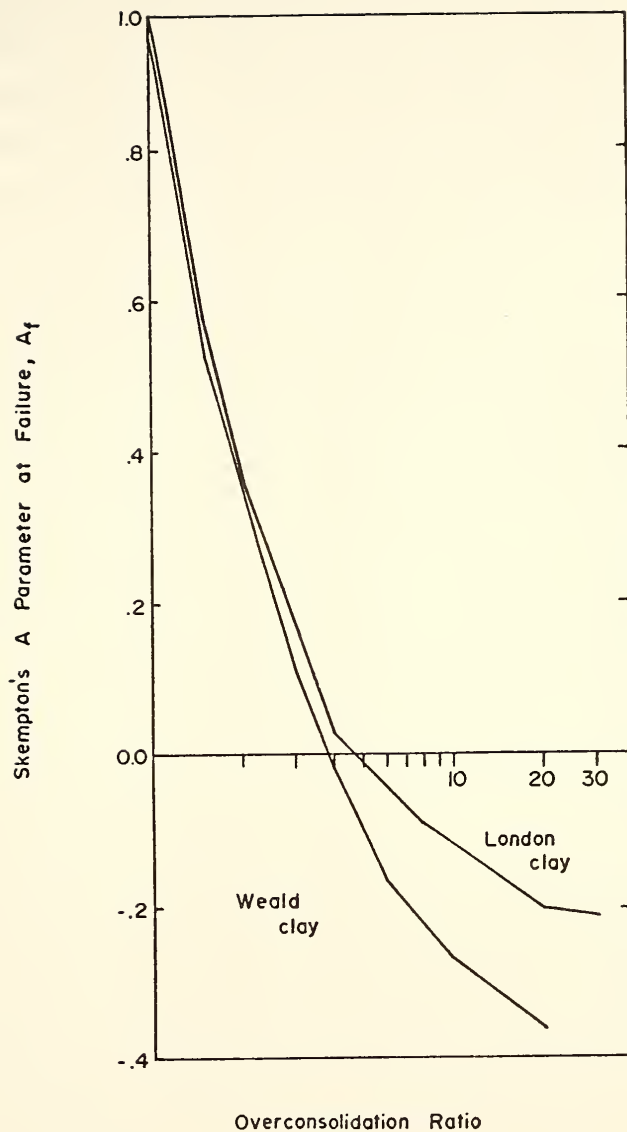


FIGURE 8 A_f VERSUS OVERCONSOLIDATION RATIO
(After Henkel, 1956)

very loose sand to -0.5 for heavily overconsolidated clay. The magnitude of the A parameter varies continuously throughout the shearing process. Figure 6 illustrates the differences in induced pore water pressure (and therefore in A) prior to failure caused by differences in soil fabric.

1-4.5 Consolidation to Anisotropic

Versus Isotropic States of Stress

Traditionally, CD and $\overline{CU}$ triaxial tests have utilized an isotropic stress state (equal all around pressure) during the consolidation phase. However, consolidation in situ occurs under the condition of limited or no lateral strain and an anisotropic stress state. The ratio of the lateral effective stress to the vertical effective stress, termed K_0 , is a function of overconsolidation ratio for a given soil with a given stress history (Brooker and Ireland, 1965). The magnitude of K_0 is commonly less than unity although values greater than one occur for very high overconsolidation ratios. Sowers, Robb, Mullis and Glenn (1957) presented data for a compacted silty clay which implied that K_0 can be greater than one following compaction if the soil is still near the surface of the embankment. To simulate in situ conditions, triaxial tests with consolidation to anisotropic stress states are now occasionally utilized.

In his statement of general strength principles for simple saturated clays, Ladd (1964) postulated that there exists a unique undrained stress path to failure in

compression and for a given overconsolidation ratio for every moisture content. This implies that the stress path of a sample consolidated under an anisotropic stress state will initiate on an intermediate point of the stress path for a sample consolidated under an isotropic stress state to the identical moisture content and then follow it to the failure envelope. This also implies a common effective stress strength envelope. Figure 9 illustrates this concept. Henkel and Sowa (1963) confirmed for normally consolidated clay that a common failure envelope exists for anisotropically and isotropically consolidated samples. However, stress paths to failure were not found to be unique for each post consolidation moisture content. For a given moisture content, anisotropic consolidation resulted in a higher shear strength than isotropic consolidation. While Ladd (1965) and Holtz and Krizek (1971) reported similar differences in stress path for the two modes of consolidation, they also noted a small increase in the effective stress strength parameters for isotropic rather than anisotropic consolidation.

The A parameter at failure is generally lower for samples consolidated under an anisotropic stress state than for those consolidated isotropically (Ladd, 1965, Holtz and Krizek, 1971, and Amerasinghe and Parry, 1975). It is therefore very difficult to predict total stress strength parameters for anisotropically consolidated samples from tests

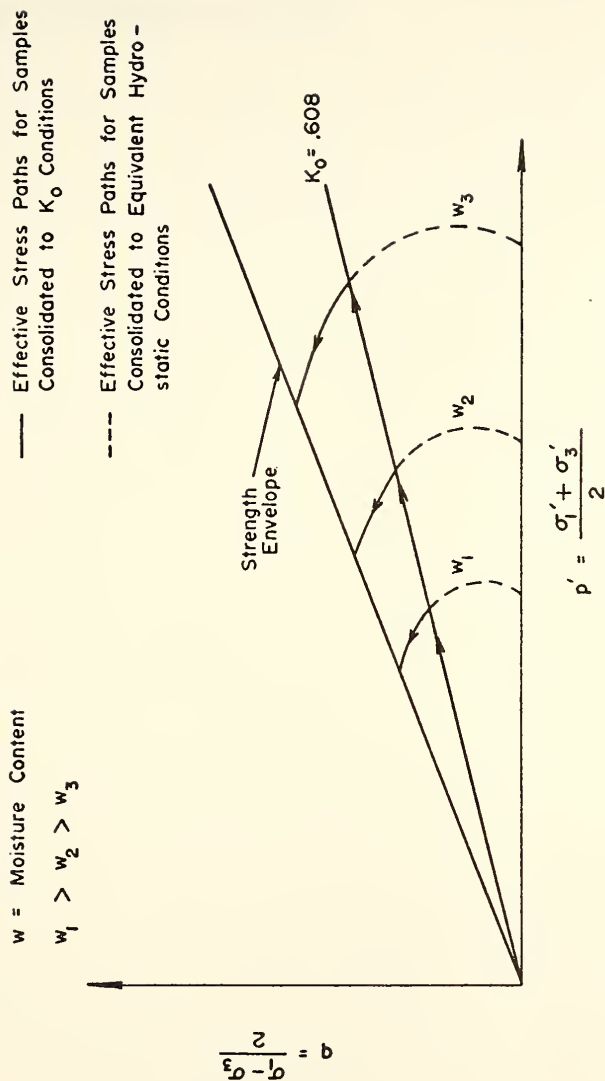


FIGURE 9 EFFECT OF K_0 CONSOLIDATION ON EFFECTIVE STRESS PATHS FOR $\overline{CU}$ TRIAXIAL TESTS ON NORMALLY CONSOLIDATED SIMPLE CLAY (After Ladd, 1964)

conducted on isotropically consolidated samples, as proposed by Lowe and Karafiath (1960).

2 - APPARATUS AND PROCEDURE

2-1 Soil Processing and Classification

The soil utilized in this study was obtained from the side of a knoll near Saint Croix, Indiana in November 1976. The site is to be a cut section in the relocation of State Road 37 being conducted by the Indiana State Highway Commission (ISHC) under construction project number F-19 (Contract number R-11178) [11]. After excavation by backhoe, the soil was shoveled into the beds of two pickup trucks, covered with plastic film to minimize moisture loss or gain, and transported to the Soil Mechanics Laboratory at Purdue University.

Until further processing the soil was stored in a large wooden bin constructed specifically for this purpose. Processing involved forcing the soil through a No. 4 sieve (4.76 mm opening) to break down the large clay lumps and remove large stones. The sieved soil was then double wrapped in large plastic bags and stored in 0.11m^3 (30 gallon) metal cans.

The results of classification tests on the soil are shown in Table 2.

The grain size distribution is shown in Figure 10. X-ray diffraction analysis revealed that kaolinite is the predominant clay mineral while traces of montmorillonite are

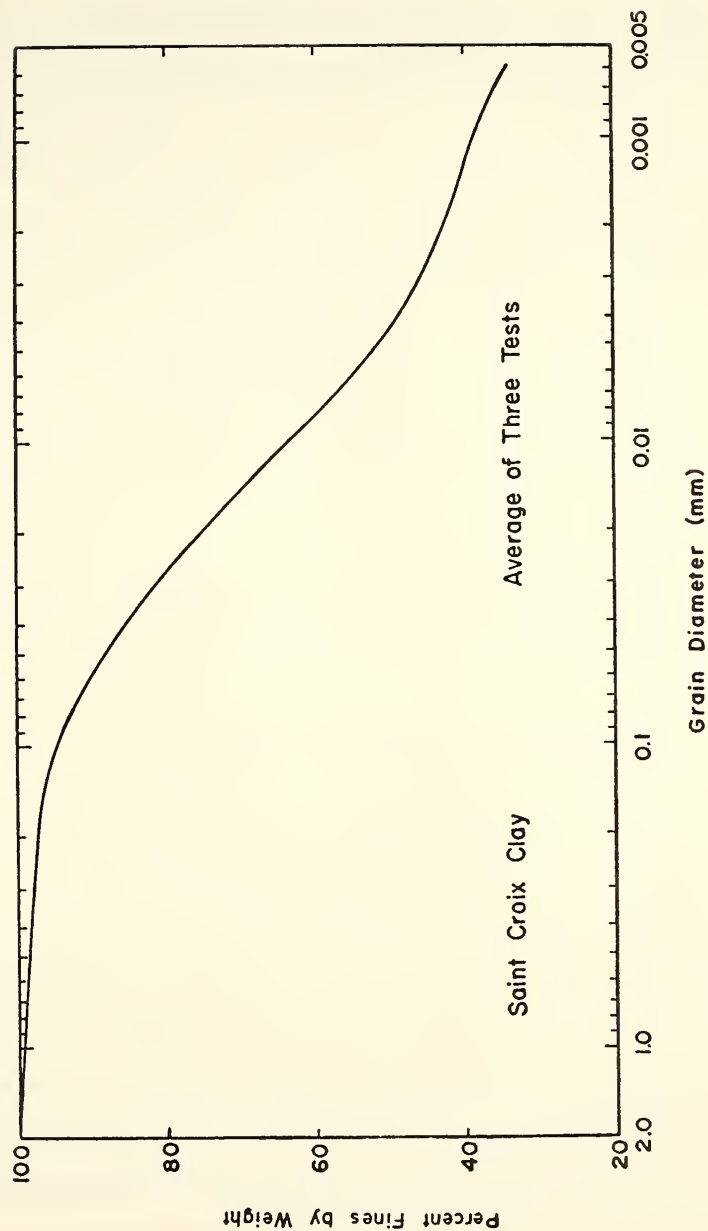
FIGURE 10
GRAIN SIZE DISTRIBUTION

TABLE 2. CLASSIFICATION TEST RESULTS FOR SAINT CROIX CLAY

Liquid Limit (%)	53
Plastic Limit (%)	22
Plasticity Index (%)	31
Natural Moisture Content (%)	18 to 20
Specific Gravity of Solids	2.80
Unified Soil Classification	CH
AASHTO Soil Classification	A-7-6

also present (Dibernardo, 1979). The parent materials of this residual clay are believed to be sandstone and shale.

2-2 Moisture Addition and Curing

Four days prior to the time of compaction, batch preparation was initiated by removing 10 to 11 kg (22 to 24 lbs) of processed soil from the lined 0.11m³ metal cans. After separating approximately 300 gms for moisture content determinations, the soil was split into four batches which were triple wrapped in plastic bags and stored in a 0.21m³ (55 gallon) covered plastic barrel for 24 hours. Approximately 25 mm (1 in) of water were kept in the bottom of the plastic barrel to insure a humid atmosphere.

At the end of the 24 hour period needed for moisture content determination, the batches were removed from the

plastic barrel and mixed together in a shallow steel pan. For the majority of the batches, achieving the desired moisture content was simply a matter of evenly distributing a predetermined amount of distilled water into the soil by means of a hand atomizer and vigorous mixing by hand. However, the batches desired at final moisture contents ranging from 13 to 19 percent were more difficult to prepare. The lined 0.11m³ metal cans used for storage of the processed soil limited evaporation so effectively that storage moisture contents ranged from 16 to 20 percent. Air drying was therefore necessary to achieve the lowest final moisture content values. Johnson and Sallberg (1961) reported that compaction after drying produces a lower dry density than compaction after wetting to the same moisture content. Therefore, the batches were dried to a moisture content slightly less than that desired and then wetted to the desired state to be consistent with the general batch preparation procedure.

It was determined from the first few batches that the desired moisture content was easily achieved by the addition of distilled water in amounts calculated to produce the desired moisture content plus one-half percent. The extra moisture was needed to replace that amount which evaporated or clung to the mixing pan. After thorough mixing of the moistened soil it was again split into four equal batches. The batches were then triple wrapped in plastic bags and

placed in the covered plastic barrel for a curing period of at least three days. The curing period creates a more uniform moisture distribution throughout the soil (Lambe, 1951).

2-3 Compaction

2-3.1 Apparatus

The California type automatic kneading compactor was utilized in this study to produce samples 101.6mm (4 in) in diameter and 116.8 mm (4.6 in) high in a standard Proctor mold. Kneading type compaction was chosen to simulate actual field compaction more closely than possible with either impact or static laboratory compaction. The kneading compactor, manufactured by the August Manufacturing Company, is shown in Figure 11. The device is electrically driven with the compactor foot pressure controlled through an air pressure gage and regulator in a pneumatic-hydraulic system. The steel tamper shoe, roughly a segment of a circle and 2064.5 square mm (3.2 square in) in area, rotates 60 degrees between each tamp to provide total coverage of the sample surface. The characteristics of kneading type compaction, summarized by Wahls, Fisher and Langfelder (1966), are a buildup of foot pressure on the sample followed by a short dwell time and pressure release, and the development of lateral shearing stresses and strains due to the limited area of load application. Figure 12 shows actual pressure versus time data for samples compacted dry of optimum (Weitzel, 1979).

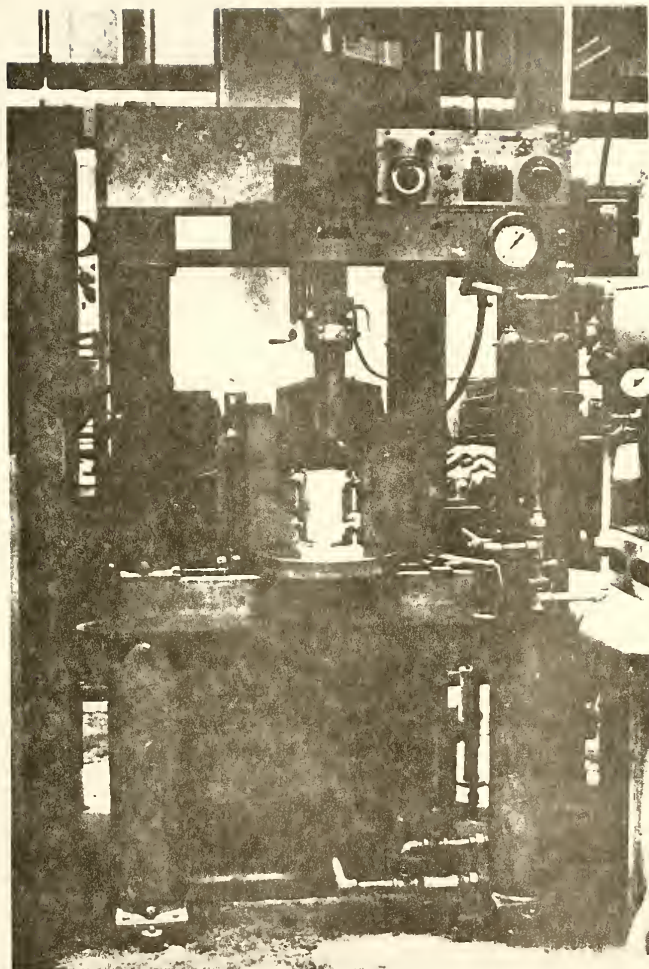


FIGURE II

CALIFORNIA KNEADING COMPACTOR

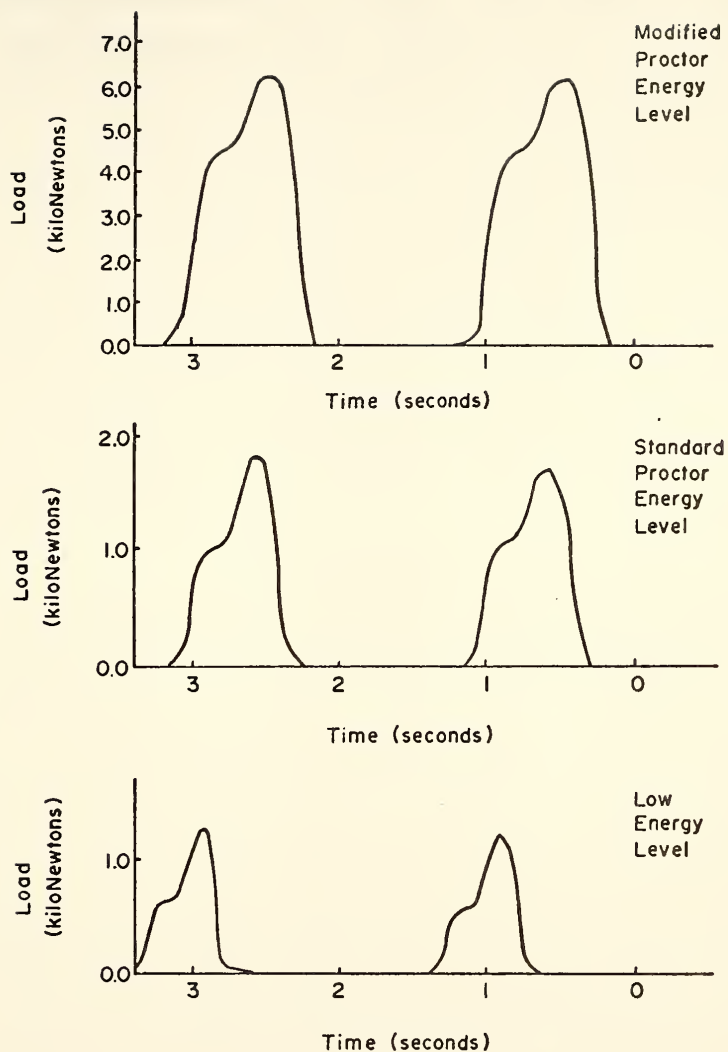


FIGURE 12 LOAD VERSUS TIME PLOTS FOR THE KNEADING COMPACTOR FOOT

Garcia-Bengochea (1978) noted that there are some limitations associated with the use of the kneading compactor. The relationship between foot pressure and gage pressure is not unique, as it appears to vary with seasonal temperature and humidity fluctuations, particularly at lower gage pressures. Comparison of the current calibration curve shown in Figure 13, with those obtained by Gaudette (1960) and Reed (1977) show significant differences. To avoid pressure fluctuations, the samples to be compacted at a given moisture content and energy level were usually compacted within the same week. The resultant sample densities were satisfactory in most cases.

2-3.2 Procedure

With the exception of variations in foot pressure, all samples were compacted in the same manner. After a minimum of three days curing time, the sample was removed from the plastic storage barrel. The compaction mold is shown assembled except for the upper collar in Figure 14. The mold is identical to that specified in AASHTO T 99-70 with the exception of a steel base shim to bring the entire sample within range of the stroke of the compactor foot. Holes were drilled in the baseplate to facilitate mounting of the mold on the compactor turntable. The soil was placed and compacted in the mold in five lifts of approximately equal thickness. The tops of the lower four lifts were scarified to insure good bond with each succeeding lift. Each lift

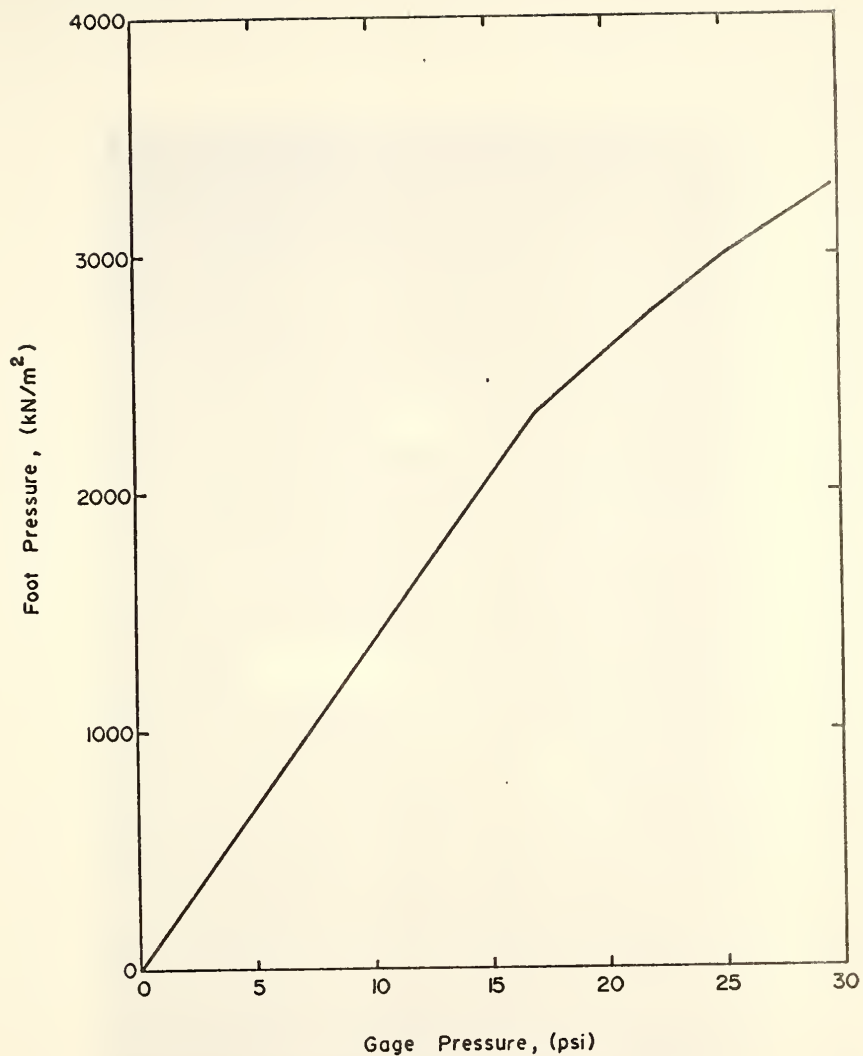


FIGURE 13 DYNAMIC FOOT PRESSURE VERSUS
GAGE PRESSURE,
CALIFORNIA KNEADING COMPACTOR

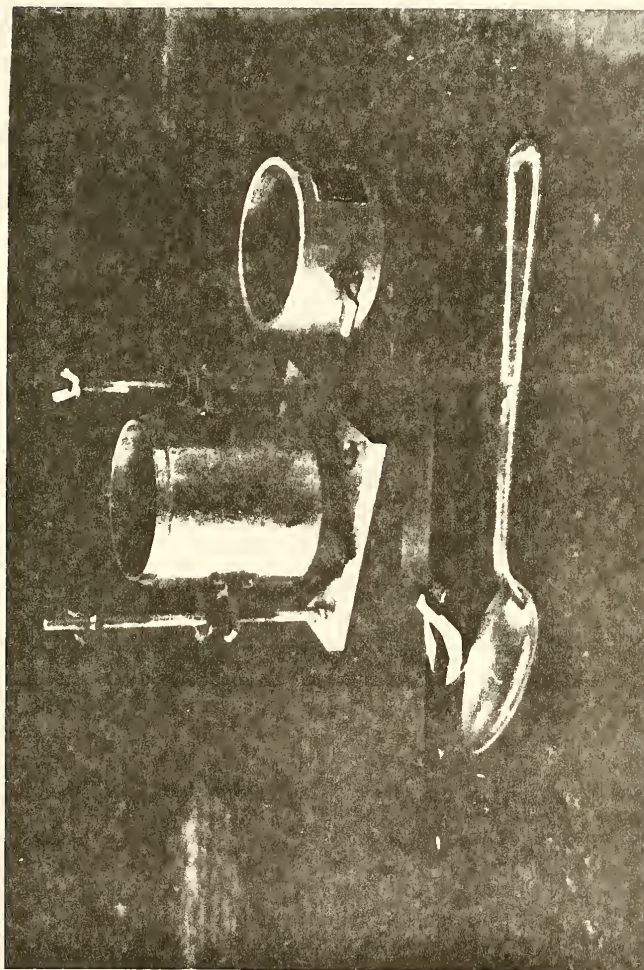


FIGURE 14 PROCTOR COMPACTION MOLD WITH KNEADING COMPACTOR
MODIFICATIONS

was subjected to thirty tamps of the compaction foot. Each series of thirty tamps was initiated with the foot at the bottom of its stroke. Garcia-Bengochea (1978) determined that initiation at other points on the cycle produced very high foot pressures during the first few strokes.

Kneading foot pressures were chosen to simulate the moisture content versus dry density relationships obtained from impact compaction tests at three energy levels. The highest energy level was the modified Proctor compaction test (AASHTO T 180-70) and the intermediate level was the standard Proctor compaction test (AASHTO T 99-70). The lowest energy level was obtained by running a standard Proctor compaction test with only fifteen blows on each of the three layers rather than twenty-five. The only major deviation from AASHTO impact compaction procedures was the use of a fresh soil sample for each compaction point.

Prior to compaction of the last lift, the compaction mold was lubricated with silicone oil to facilitate separation of the collar from the mold and prevent damage to the sample. After compaction of the final lift, the mold collar was carefully removed and the excess soil above the mold rim was trimmed away with a sharpened straight edge. The finished compaction sample and mold were then weighed to within 0.1 gm.

2-4 Sampling

2-4.1 High and Intermediate

Sample Densities

After weighing the compacted sample and mold for a density determination, the sample was extruded from the compaction mold using a hydraulic jack and a reaction plate with an orifice 101.6 mm (4 in) in diameter. The extruded sample was then placed in a split mold which was tightened in preparation for sampling. Three samples, 35.6 mm (1.4 in) in diameter, were desired from each large sample of compacted soil. Sharpened, thin-walled stainless steel tubes (similar to those used by Scott (1977)) were used exclusively for the high and intermediate density samples, as the soil strength was too great for wire saw trimming. A length to diameter ratio of at least two for each sample was desired to minimize the effects of end restraint created by the loading platens during triaxial testing (Bowles, 1970). The sampling tube dimensions, 81.3 mm (3.2 in) long, 38.1 mm (1.5 in) in outside diameter and a wall thickness of 2.5 mm (0.098 in), were determined on the basis of a two to one aspect ratio, with room to spare for end trimming.

Figure 15 shows the sampling process. Three sampling tubes, lubricated with silicone oil, were simultaneously pressed into the compacted sample using the hydraulic jack mentioned previously. To accommodate the volume of the advancing sampling tubes, the split mold clamps were

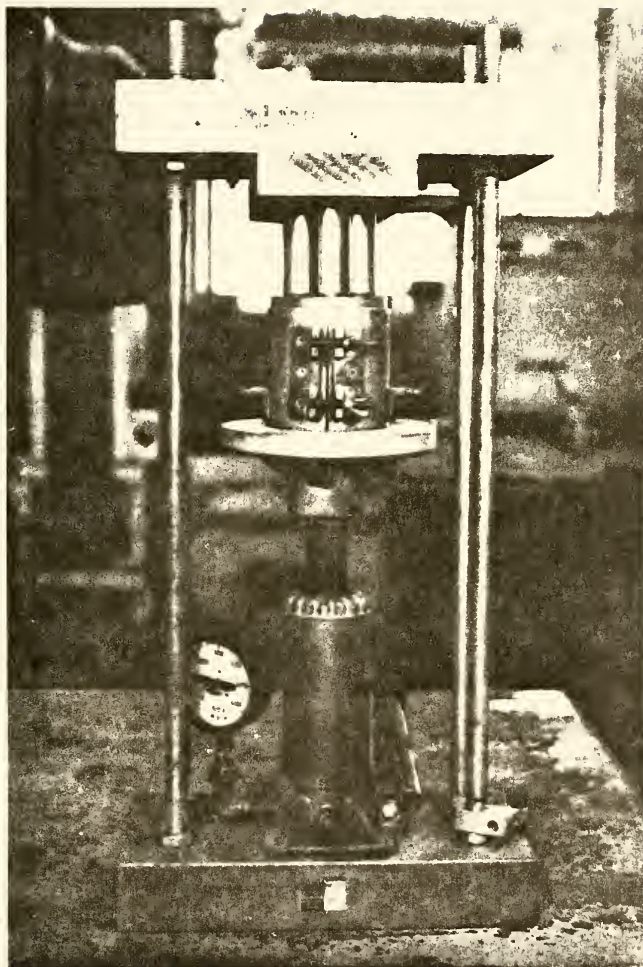


FIGURE 15 SAMPLING METHOD FOR DENSE
COMPACTION SPECIMENS

loosened as necessary to allow lateral expansion of the mold. To retrieve the tube-encased samples, it was necessary to completely loosen the clamps of the split mold, extrude the sample from the mold with the hydraulic jack and break the compacted sample apart. The soil between the sampling tubes was used for three moisture content determinations. Figure 16 shows the special attachments to the hydraulic jack used for extruding the samples from the sampling tubes. The circular plate is an orifice reducer for the jack's reaction plate and the column is the extrusion piston. After extrusion the samples were trimmed to the desired length as right circular cylinders, measured three times for height and six times for diameter, and weighed. The finished samples were then enclosed individually in small plastic bags and stored in a high humidity jar awaiting completion of the triaxial cell set-up.

2-4.2 Low Sample Densities

The sampling procedure used for the high and intermediate density samples was found to be too strenuous for the low density samples, as sample disturbance was evinced by significant density increases during sampling. Wire saw trimming, commonly employed with soft clays, was found to be perfectly suited for the low density samples. After extrusion of the compacted sample from the compaction mold it was placed in a mold with slits which allowed the sample to be quartered carefully with a hacksaw. Each quarter was placed

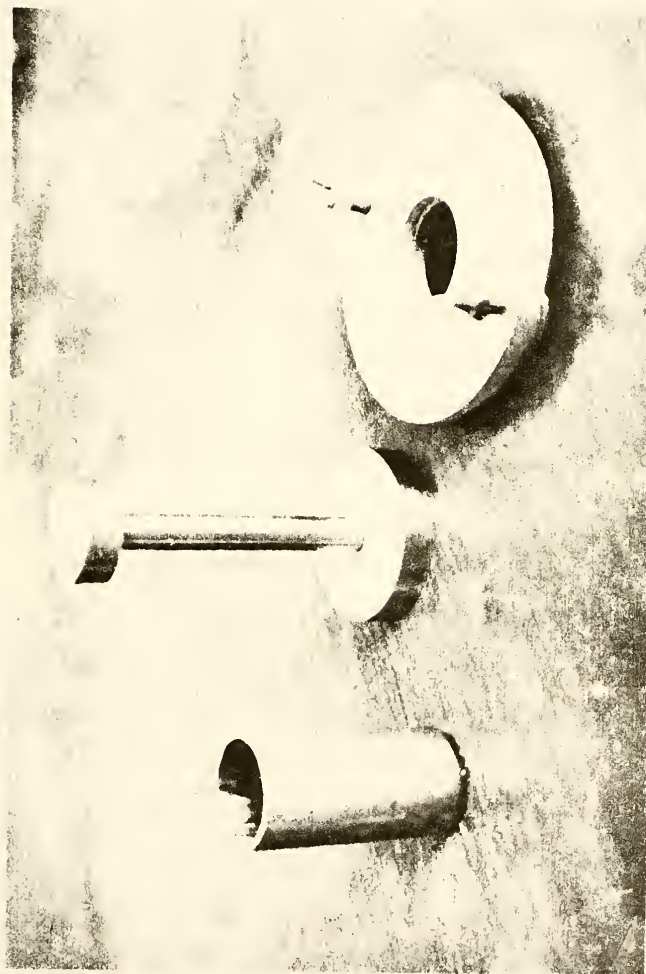


FIGURE 16 ATTACHMENTS FOR SAMPLE EXTRUSION

in the trimming frame shown in Figure 17 and trimmed to approximately 35.6mm (1.4in) in diameter with a wire saw. The remainder of the procedure is identical to that employed for the samples obtained with sampling tubes.

2-5 Sample Set-up

Sample set-up was usually initiated from 1/2 to three days after compaction to minimize the effects of thixotropy on swell and consolidation (Seed and Chan, 1957 and Gray and Kashmeeri, 1971). Figure 18 shows a disassembled triaxial cell. Triaxial cell bases were initially submerged in a water bath and flushed thoroughly to remove all entrapped air. The pore pressure transducers were then attached, all valves were closed and the bases were removed from the bath. The best two of the three tube samples, or the best two of the four wire saw samples, were then removed from the high humidity jar. A minimal difference between small and large sample densities was the quality criterion. The samples were wrapped with longitudinal filter paper drains to shorten the drainage path and accelerate back pressure saturation, consolidation, and pore pressure equalization (Bishop and Henkel, 1962). Soft filter paper was used (Whatman No. 1) to minimize the effect on sample strength (Olson and Kiefer, 1963). Filter paper and saturated porous stone discs were placed on both ends of the samples which were then seated on the triaxial cell base pedestals. Employing a membrane stretcher, each sample was enclosed in two rubber membranes

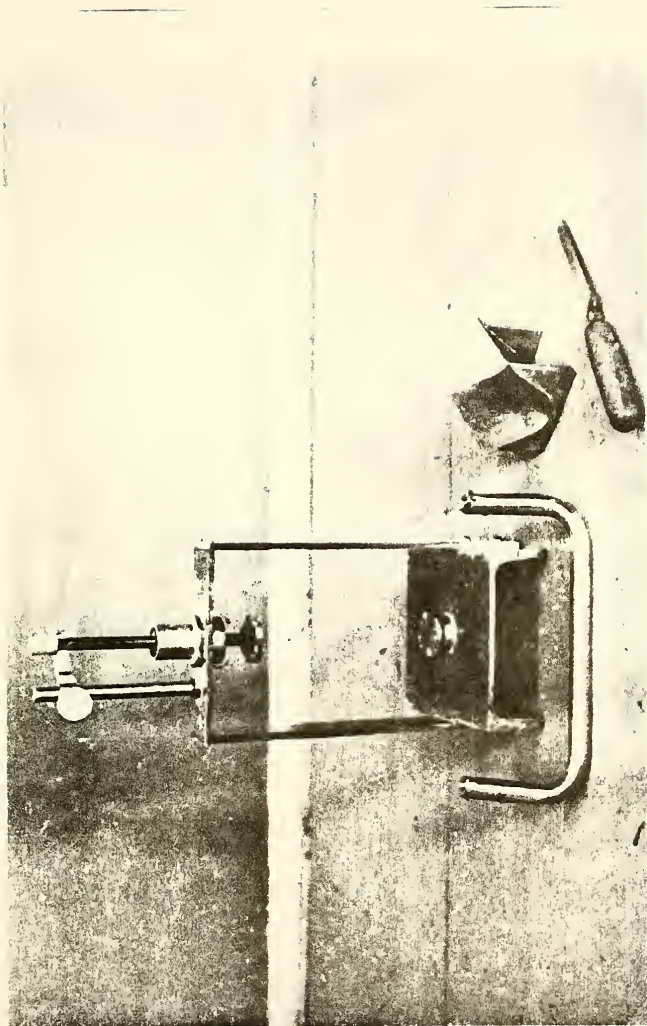


FIGURE 17 WIRE SAW TRIMMING APPARATUS

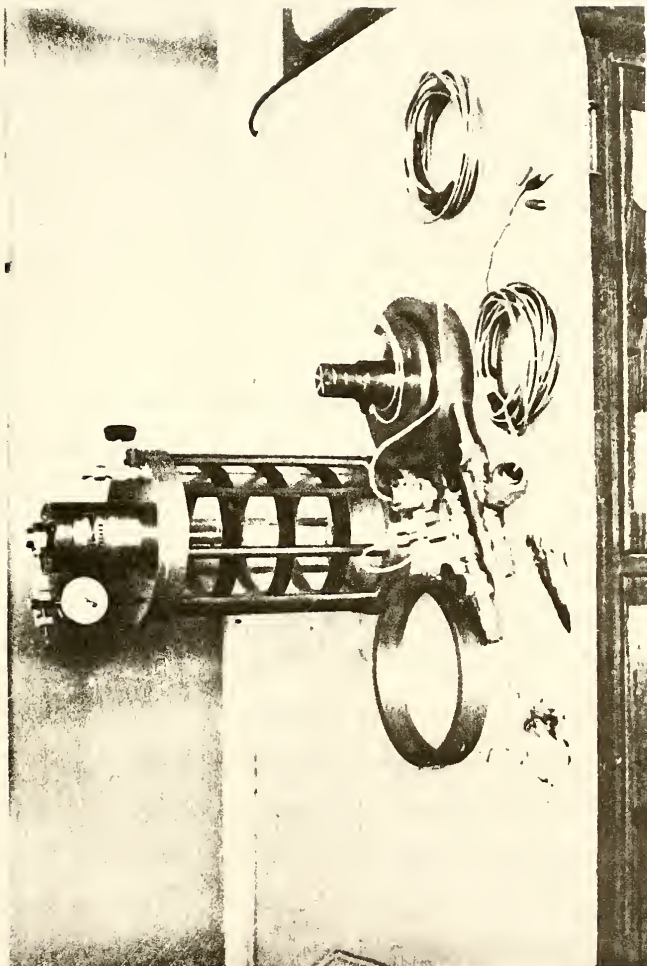
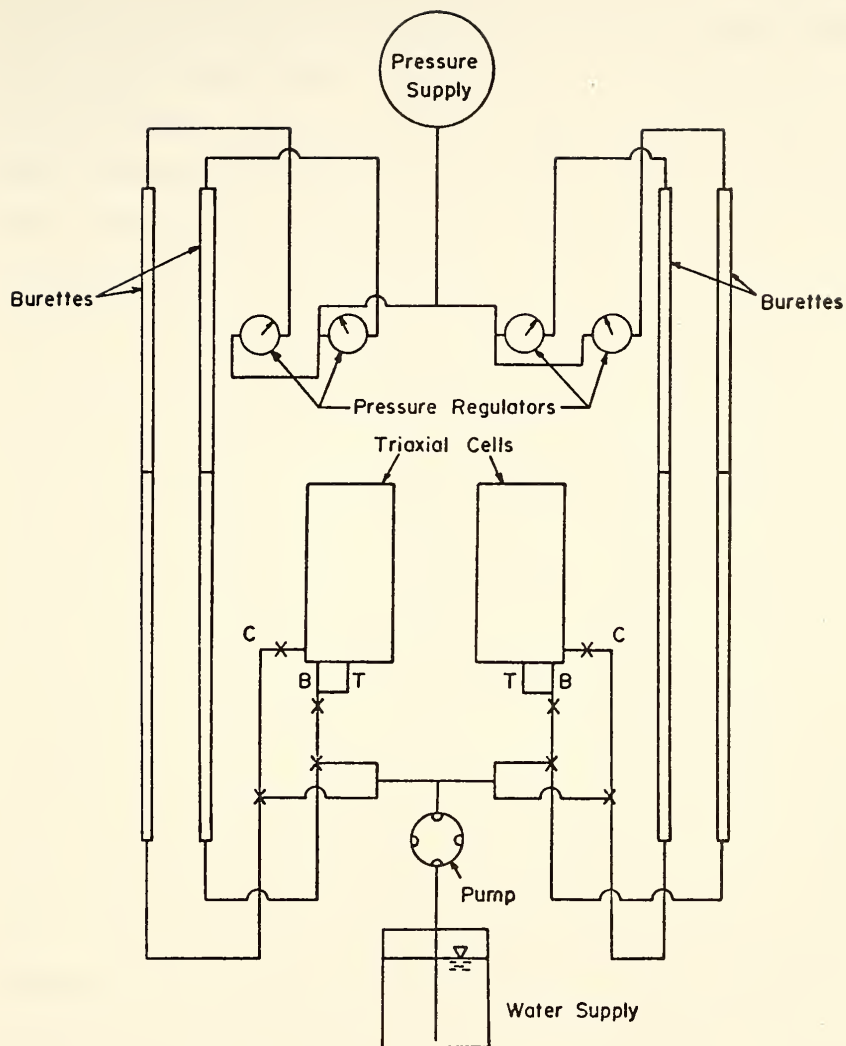


FIGURE 18 DISASSEMBLED TRIAXIAL CELL

having a combined thickness of 0.102mm. Two membranes were used to protect against pinhole leaks. O-rings were used to provide a leak-proof seal between the rubber membranes and the loading caps and base pedestals. The triaxial cell chambers were then positioned, clamped and filled with distilled, deaired water. During the filling operation, the loading pistons were lowered to contact with the loading cap bearings and the dial gage readings (to 0.0254 mm) were taken to correspond to initial sample heights. After recording the gage readings the loading pistons were raised and locked in place.

2-6 Back Pressure Saturation

A schematic of the saturation and consolidation system is shown in Figure 19. Total pressure supply was limited by the 1034 kN/m^2 (150 psi) capacity of the air compressor. Consolidation pressures of 69, 138 and 276 kN/m^2 (10, 20 and 40 psi) left 965, 896 and 758 kN/m^2 (140, 130 and 110 psi) respectively available for back pressure. Lowe and Johnson (1960) indicated that higher pressures would perhaps be needed for initial compaction conditions with low levels of saturation. To minimize the required back pressure it was necessary to remove as much air as possible from inside the samples and all air trapped around the samples prior to back pressure application.



C = Cell Pressure
 T = Top Back Pressure Line
 B = Bottom Back Pressure Line

FIGURE 19 SCHEMATIC OF THE SATURATION—
 CONSOLIDATION SYSTEM

A supply of deaired distilled water was supplied from a 6.35 mm (0.25 in) diameter burette to each sample through the base pedestal back pressure line. A vacuum was applied to the loading cap back pressure line. Simultaneously a cell pressure was applied such that the combined vacuum and cell pressure totalled less than 69 kN/m^2 (10 psi), the minimum final consolidation pressure. Approximately once every 15 minutes the pressure in the base pedestal back pressure line was raised slightly above the overall confining pressure to balloon the membrane and flush the accumulation of trapped air out through the loading cap back pressure line (Black and Lee, 1973). In this manner, air leached from the samples and air trapped during membrane placement were removed. This process was continued until the flow during flushing was clear of air bubbles. This generally required one or two hours, although a few samples emitted air bubbles for as long as six hours. When flushing was complete the vacuum was removed from the loading cap back pressure line, which was then reconnected to the back pressure loop. Flow through the back pressure system (from the base pedestal through the top cap) was maintained during this procedure to prevent air entrapment.

Following flushing and prior to back pressure application it was generally necessary to realign the samples with the loading caps and the base pedestals. The triaxial cells were detached from the saturation and consolidation system and all valves were closed. After removing the cell

chambers the samples were realigned and the back pressure lines were observed to note the presence of any air bubbles. The chambers were then positioned, clamped and filled with deaired, distilled water. If bubbles were present the flushing procedure was continued. Otherwise, the triaxial cells were simply reconnected to the saturation and consolidation system.

The next step in the procedure was the application of pressure to the pore fluid to reduce the volume of the retained air and force some of it into solution. The procedure for simultaneously increasing the cell pressure and back pressure to achieve saturation without excessive cyclic loading of the samples is described by Lowe and Johnson (1960). For final consolidation pressures of 138 kN/m^2 (20 psi) or 276 kN/m^2 (40 psi), the cell pressure and back pressure were incremented by 69 kN/m^2 (10 psi) after each two hour pressure equalization period. A differential of 69 kN/m^2 (10 psi) was maintained between them. The pressures were incremented until saturation was achieved. For the final consolidation pressure of 69 kN/m^2 (10 psi) the differential between the cell pressure and the back pressure was 21 kN/m^2 (3 psi), while both were incremented by 48 kN/m^2 (7 psi) at two hour intervals.

Two methods of checking saturation, one quantitative and one qualitative, were employed during back pressuring. The quantitative method is a B parameter check. The B parameter was defined by Skempton (1954) as the ratio of the

change in pore pressure to the change in cell pressure which induced it. This check was made prior to application of each increment of cell and back pressure. After closing the back pressure line a known increment of cell pressure was applied and the pore fluid response after one minute was recorded. An equal increment in back pressure was then applied after opening the back pressure line. Although a B value of unity is commonly considered to represent saturation, the B parameter is actually a function of the soil porosity and the relative compressibilities of the soil structure and pore fluid (Skempton, 1965). Final B parameter values obtained in this study ranged from 0.7 to 1.0 with 90 percent greater than or equal to 0.9. The qualitative check for saturation involved monitoring the water level in the back pressure line after closing the cell pressure line and increasing the back pressure (Lee, Morrison and Haley, 1969 and Abeyesekera, 1978). A sudden dip in the water level in the back pressure burette evinced the compression of air within the back pressuring system. Samples were assumed to be saturated when water level changes were small and the B parameter value remained constant for three consecutive back and cell pressure increments (Lee, Morrison and Haley, 1969 and Wissa, 1969).

2-7 Consolidation

After saturation had been achieved, generally within the thirty hours following set-up, the back pressure was

maintained constant while the cell pressure was increased to create a differential equal to the desired final consolidation pressure (69, 138, 207 or 276 kN/m^2) (10,20,30 or 40 psi). Opening the back pressure lines initiated the expulsion of water from each specimen. The consolidation process was monitored by recording the amount of water flow from each sample until primary consolidation was complete and secondary compression was evidenced. The end of primary consolidation was determined by Casagrande's logarithm of time method (Perloff and Baron, 1976) and required from 0.5 to 9 hours.

Four samples were consolidated under an anisotropic stress state to a ratio of lateral effective stress to vertical effective stress of 0.67. The axial load in excess of that applied by the cell pressure was supplied either by a calibrated, air pressure controlled loading piston or through a proving ring by careful manipulation of the strain rate applied in an axial loading frame. The rest of the consolidation procedure was identical to that used for consolidation to an isotropic stress state.

2-8 Undrained Shear

Following consolidation the valve to the back pressure line was closed and the sample was axially compressed at a constant rate of strain in a gear driven loading frame produced by Engineering Laboratory Equipment Ltd. The cell

pressure was maintained at a constant level during compression by monitoring with a calibrated Tyco model AB pressure transducer (1379 kN/m^2 or 200 psi capacity; 1 kN/m^2 or 0.1 psi sensitivity) and adjusting with a Kendall model 10 air pressure regulator (1379 kN/m^2 or 200 psi capacity). Axial load was measured with a calibrated proving ring while axial deformation was monitored with a dial gage (sensitivity of 0.0254mm or 0.001 in) attached to the loading piston and in contact with the triaxial chamber cap. Axial load due to water pressure on the piston and bushing friction was deducted from the measured load by taking the proving ring reading during downward movement, just prior to contact of the piston and loading ball, as zero axial load. Pore water pressures were measured with a calibrated Tyco model AB pressure transducer attached to the triaxial cell base with access to both the top and bottom of the sample. Cell and pore pressure transducers were calibrated and monitored with Hewlett Packard model 3476A Digital Multimeters. Axial compression was continued until slightly past the load peak or to twenty percent strain if no peak occurred.

It was necessary to select a strain rate which allowed pore water pressure equalization throughout the sample volume. It was also desirable to use a strain rate which could be matched by the equipment being used for a companion triaxial study being conducted in the unconsolidated undrained mode (Weitzel, 1979). A deformation rate of 0.0635 mm (0.0025 in) per minute was chosen, which translates to

approximately 0.09 percent strain per minute. At this rate, 20 percent strain is achieved in 3.5 to 4 hours. Results published by Chan and Rivard (1963), Bazett and Bell (1963) and Bishop and Henkel (1962) implied that the chosen deformation rate was perhaps too rapid for complete pore pressure equalization. A check was made for each test using the method proposed by Blight (1963) which requires knowledge of the sample dimensions and drainage conditions, the coefficient of consolidation (c_v) and the desired percentage of pore pressure equalization. The coefficient of consolidation was calculated from the consolidation data of each test using the square root of time method discussed by Bishop and Henkel (1962). Some 17 percent of the tests were calculated to have attained less than 90 and more than 80 percent pore pressure equalization. These were checked for significantly different A parameter values and poor strength envelope fit as a basis for accepting or rejecting them. None were rejected. It must be noted however that the chosen rate of testing makes interpretation of stress paths difficult, as the early portion of each test suffered the steepest pore pressure gradient throughout the sample with insufficient time for equalization (Blight, 1963).

2-9 Final Void Ratio Determination

Following undrained shear the valves to the back pressure and cell pressure burettes were closed, pressures backed down and the cell was removed from the saturation and

consolidation system. The triaxial cell was quickly broken down and the sample removed. Rubber membranes, filter paper and porous stones were removed and the sample was weighed in air on a Mettler P 1210 balance (0.01 gm accuracy). The sample was then hand dipped in melted wax until it was thoroughly coated. The waxed sample was weighed in air and then in water in a submerged basket hung beneath the Mettler balance. Archimedes' principle was used to calculate the sample volume, knowing that the specific gravity of the wax was 0.805 (Leonards, 1955 and Scott, 1977). The wax was then peeled from the sample and a final water content determination was made using the whole sample. The final percentage saturation was calculated, knowing the specific gravity of the soil, the final water content and the void ratio.

2-10 Summary Listing of Independent Variables

A. Materials and Processing

1. Soil: 1 type; A-7-6
2. Soil Mineralogy: 1 type determined by X-ray diffraction
3. Processing: 1 level; passing No. 4 sieve at natural moisture content

B. Compaction

1. Compactive Technique: 1 type, kneading; 5 lifts, sample volume equals 0.00094 cubic meters
2. Compactive Effort: 3 levels; kneading foot pressure adjusted to attain densities on high, medium and low energy Proctor curves.

3. Moisture Content: 3 levels; dry of optimum, optimum, wet of optimum

C. Sampling: 2 techniques: small samples cut from 0.00094 cubic meter sample. Sampling tubes (high and intermediate densities) or wire saw (low densities) used.

D. Saturation: 1 technique; back pressure saturation

E. Consolidation

1. Pressure: 3 levels; isotropic consolidation, to net confining pressures of 69, 138 and 276 kN/m². A limited number of tests were consolidated anisotropically for comparison.

2. Sample Volume Determination: 1 technique, waxed samples weighed in air and in water following shear.

F. Shear Testing

1. Testing Technique: 1 type; axial compression at constant rate of strain, constant cell pressure, axisymmetric stress system.

2. Limiting Test Condition: 1 type; 20 percent axial strain or maximum ($\sigma_1 - \sigma_3$)

3. Pore Pressure Measurement: 1 technique; pore pressure transducer.

4. Axial Load Measurement: 1 technique; proving ring.

3 - RESULTS AND DISCUSSION OF RESULTS

3-1 Compaction

3-1.1 Dry Density Versus Moisture Content Relationships

The dry density versus moisture content relationships utilized in this study were determined from impact compaction tests at three levels of gross input energy. The test specifications for each energy level are listed in Table 3. The impact compaction data are listed in Appendix A. The statistical relationships between dry density and moisture content obtained from least squares analysis of these data are shown in Figure 20. Satisfactory fit to the data is indicated by adjusted coefficients of multiple determination (R_a^2) of 0.90, 0.92 and 0.83 respectively for modified Proctor, standard Proctor and low energy Proctor.

A California type kneading compactor was utilized to compact the samples to be sheared in this study, as kneading compaction simulates field compaction more closely than static or impact compaction. To attain dry density values on the impact compaction curves for each moisture content it was necessary to compact samples with different foot pressures until the desired dry density was achieved.

TABLE 3. SPECIFICATIONS FOR IMPACT COMPACTION TESTS

Test Series	Volume Of Mold(m ³)	Height Of Hammer Drop (mm)	Number Of Layers	Weight Of Hammer (Newtons)	Number Of Blows Per Layer	Gross Energy (kJ/m ³)
Modified Proctor	0.000944 (1/30 ft ³)	457 (18 in)	5	44.5 (10 lbs)	25	2693 (56,250 ft-lb/ft ³)
Standard Proctor	0.000944 (1/30 ft ³)	305 (12 in)	3	24.5 (5.5 lbs)	25	594 (12,400 ft-lb/ft ³)
Low Energy Proctor	0.000944 (1/30 ft ³)	305 (12 in)	3	24.5 (5.5 lbs)	15	356 (7,400 ft-lb/ft ³)

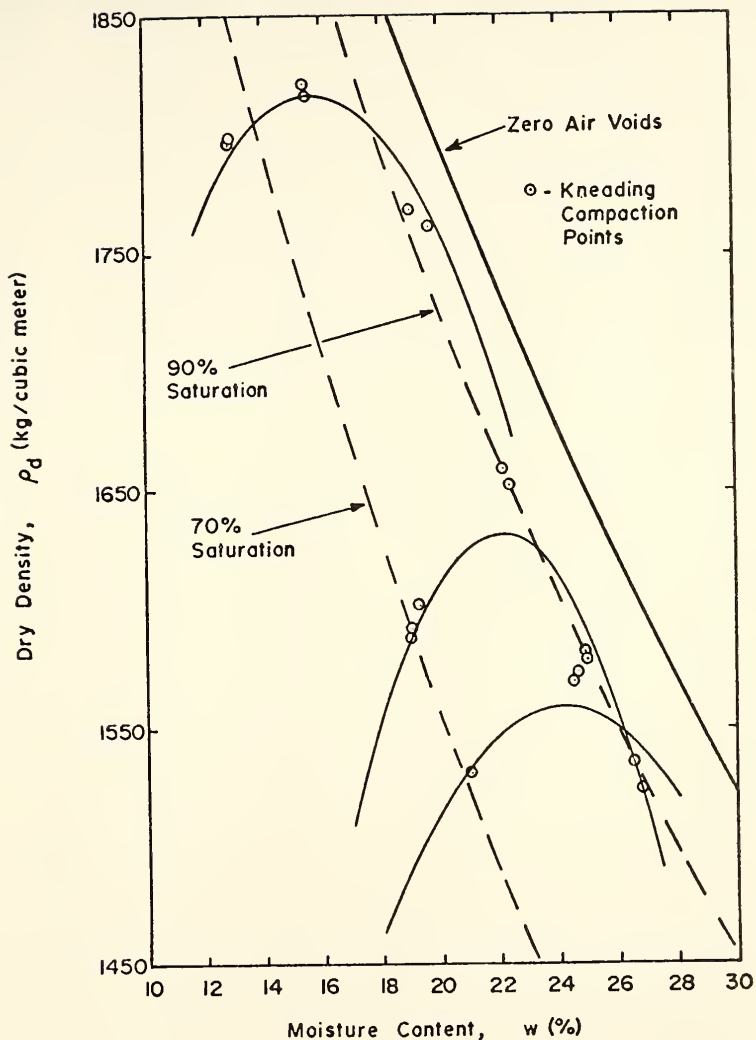


FIGURE 20 DRY DENSITY VERSUS MOISTURE CONTENT CURVES FROM IMPACT COMPACTION TESTS

Figure 21 shows the dry density versus moisture content relationships achieved with different foot pressures and compares them to those obtained from impact compaction. The kneading compaction dry density and moisture content points utilized in the shearing program are shown in Figure 20. Table B1 lists the compactive foot pressures utilized for each point. Moisture contents ranged from 3 percent dry of optimum to 3 percent wet of optimum at each energy level and effectively bracket the probable range of compaction conditions.

3-1.2 Definition of Compaction Variables

Dry density, moisture content and compaction energy are termed compaction variables. The purpose of this study is to relate the effective stress strength parameters, Skempton's A parameter at failure and volume change due to saturation and consolidation to these compaction variables. It was therefore vital that a consistent, accurate procedure be utilized for defining and measuring the compaction variables. For this study the compaction variables were defined by measurements made on the 0.000944 m^3 ($1/30 \text{ ft}^3$) samples formed in the compaction mold (hereafter referred to as the large samples) rather than on the individual test samples cut from each large sample (hereafter referred to as the small samples).

Moisture content (w, %) was determined by averaging the results of three separate 100 gm moisture content

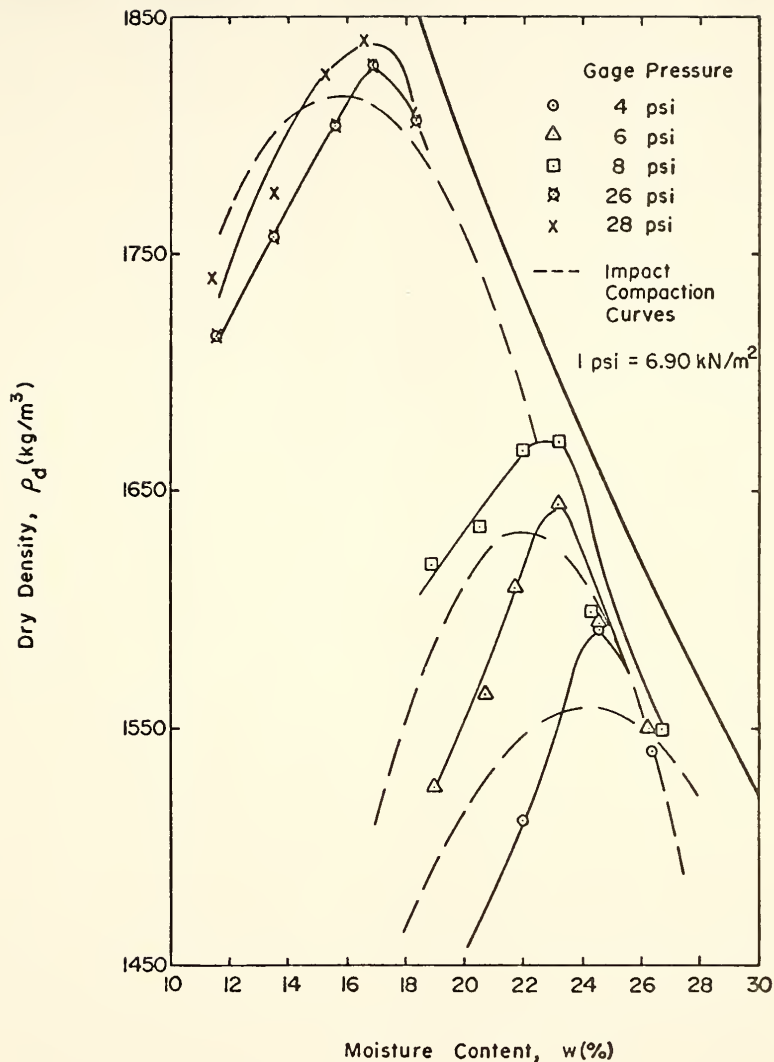


FIGURE 21 DRY DENSITY VERSUS MOISTURE CONTENT CURVES FROM KNEADING COMPACTION TESTS

samples taken along the longitudinal axis of the large sample rather than relying on trimmings from each small sample. Comparison of the two measurement procedures revealed that small sample trimmings resulted in moisture contents consistently lower by 0.1 or 0.2 percent. This difference was attributed to moisture lost due to the high surface area to volume ratio for small sample trimmings during the short but unavoidable exposure to the atmosphere. Large sample moisture contents were therefore reported to achieve greater consistency and are listed in Table B1.

Dry density (ρ_d , kg/m^3) was calculated from the measured values of wet density (ρ , kg/m^3) and average large sample moisture content. The wet density was obtained by weighing the large sample and dividing by its volume (0.000944 m^3). The density of each small sample was also determined by dividing its net weight by the volume calculated from the measured sample dimensions. Small sample and large sample densities are both listed in Table B1. For compaction wet of optimum the large and small sample dry densities are very similar. However, for the dry of optimum specimens the small sample dry densities are significantly lower than those for the large samples from which they were taken. This difference is attributed to the surface roughness of the small samples after sampling and trimming. As discussed in section 1-3, soil compacted dry of optimum retains much of the aggregate level fabric present prior to

compaction and acts in a brittle manner. These aggregates resist shear during sampling and trimming and are often pulled intact from the sample surface. Calculation of sample volume from measured sample dimensions does not take account of the numerous pockmarks caused by aggregate pull-out. The calculated volumes are too large and result in low small sample densities. Therefore the dry density values obtained from the large samples represent the compaction conditions more accurately and consistently.

The maximum pressure applied by the kneading foot during each stroke (p_c , kN/m^2) was utilized as a measure of compaction energy. DiBernardo (1979) utilized this measure in a study of the compressibility characteristics of a compacted highly plastic clay and determined that both compactive prestress and percent volume change upon wetting correlate well with it.

3-1.3 Sample Nomenclature

Each sample tested in this study was assigned a name based on compaction conditions and consolidation pressure. First, each sample was assigned a letter to signify to which dry density versus moisture content curve it corresponded. M, S and L stand for modified Proctor, standard Proctor and low energy Proctor respectively. Next, a D, O or W was assigned to signify compaction dry, at or wet of optimum, respectively. A number from 1 to 7 was then assigned to differentiate each sample from the others with the identical

compaction conditions. Last of all, the consolidation pressure in kN/m^2 was enclosed in parentheses and placed after the first three symbols. For example, the first sample compacted dry of optimum on the modified Proctor curve and consolidated to 69 kN/m^2 would appear as MD1(69). If the value of the consolidation pressure is not needed, it may be abbreviated to MD1. The complete series of tests for a given compaction condition may be abbreviated to MD.

3-2 Saturation

The initial and final degrees of saturation for each of the samples tested during this study are listed in Table 4. Each initial degree of saturation was obtained through calculations utilizing the specific gravity of Saint Croix clay and the appropriate values of as-compacted moisture content and dry density. The final degrees of saturation were obtained through calculations which utilized the specific gravity and the post-shear values of moisture content and dry density determined after weighing the waxed samples in air and in water.

The final degree of saturation values obtained during the course of this study range from 95.5 to 100.0 percent. They are plotted versus the appropriate initial degree of saturation values in Figure 22. It is difficult to determine the degree of saturation below which the soil is partially saturated and above which it is saturated for all practical purposes. Lowe and Johnson (1960) stated that samples at 99 percent saturation may test no differently than those

TABLE 4. BACK PRESSURES REQUIRED TO ATTAIN THE EXPERIMENTALLY OBSERVED DEGREES OF SATURATION

Sample No.	S_i (%)	S_f (%)	Theoretical Back Pressure Δu_T^* (kN/m ²)	Experimental Back Pressure Δu_E (kN/m ²)	Final B Parameter
MD1 (69)	65.1	97.9	839	483	0.94
MD2 (138)	64.4	98.4	1009	759	0.94
MD3 (276)	64.4	99.1	1305	828	0.92
MD4 (276)	65.1	98.9	1169	690	1.00
MO1 (69)	80.6	97.8	433	386	1.00
MO2 (138)	80.3	98.5	552	621	0.98
MO3 (276)	80.3	97.5	406	759	0.94
MO4 (69)	80.6	99.0	646	386	0.91
MW1 (85)	93.5	99.8	311	338	0.95
MW2 (138)	91.3	96.0	82	621	0.96
MW3 (276)	91.4	97.0	117	690	0.98
MW4 (76)	91.3	95.7	72	372	0.96
MW5 (207)	91.4	96.6	100	552	0.98
MW6 (276)	93.5	99.7	288	483	1.00
SD1 (69)	70.2	95.5	406	418	0.80
SD2 (138)	72.4	97.6	600	690	0.96
SD3 (276)	69.8	99.4	1241	690	1.00
SD4 (138)	69.8	98.8	985	621	0.94
SD5 (207)	72.4	97.6	600	759	0.92
SD6 (276)	70.2	97.4	623	793	0.70
SD7 (138)	70.2	96.6	511	579	0.85
SO1 (69)	90.7	97.2	145	579	0.88
SO2 (138)	90.2	96.0	100	828	0.96
SO3 (276)	90.2	97.0	144	815	0.94
SO4 (239)	90.2	96.3	110	724	0.98
SW1 (69)	90.3	97.1	145	483	1.00
SW2 (138)	90.3	97.9	196	690	0.98
SW3 (276)	90.6	98.7	261	690	0.90
SW4 (138)	90.6	97.1	140	621	0.98
LD1 (69)	70.9	98.3	775	531	0.97
LD2 (138)	70.9	98.1	729	552	0.98
LD3 (276)	70.9	98.6	867	690	0.96
LD4 (69)	70.9	100.0	1566	531	1.00
LO1 (69)	87.5	98.2	299	338	1.00
LO2 (138)	88.6	97.3	196	552	0.98
LO3 (276)	88.6	99.1	383	690	0.90
LO4 (69)	88.6	98.4	288	435	0.97
LO5 (138)	87.5	98.1	291	690	0.92
LO6 (276)	88.6	99.4	440	690	0.98
LW1 (69)	89.7	99.1	347	338	1.00
LW2 (138)	89.7	98.3	248	483	0.96
LW3 (276)	89.9	97.4	170	690	0.94

* Calculated from equation (1-2) after Lowe and Johnson (1960).

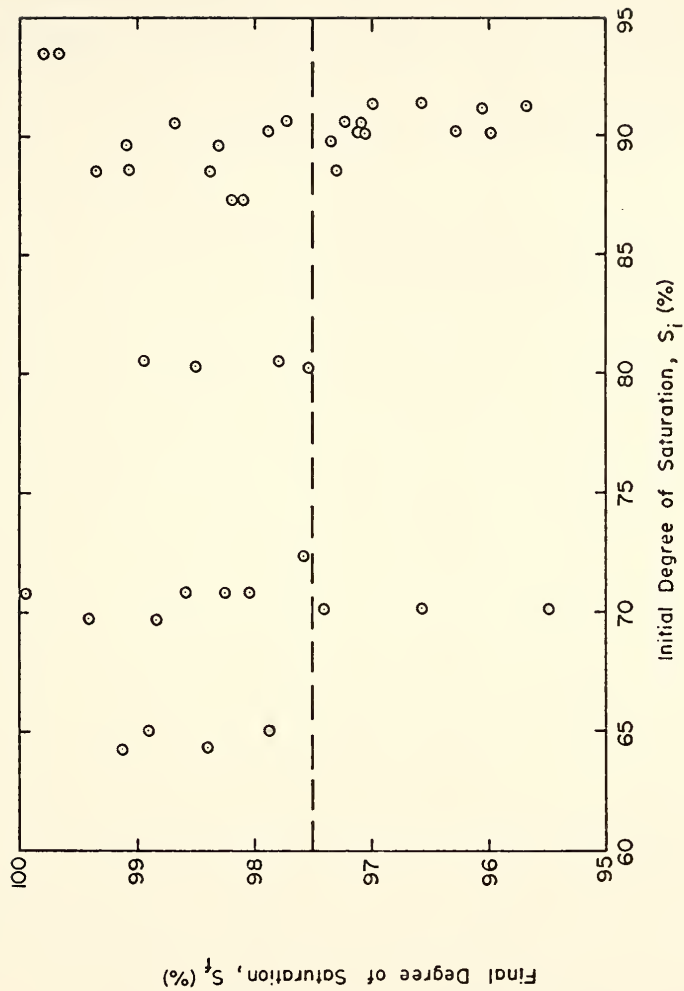


FIGURE 22 COMPARISON OF INITIAL AND FINAL VALUES OF DEGREE OF SATURATION

tested at 100 percent saturation. However, it is almost certain that samples which are only 95 percent saturated will undergo significant volume changes during shear, classifying them as partially saturated. As the majority of the final degree of saturation values obtained in this study lie between 95 and 99 percent, further clarification and discussion is necessary.

All of the reported values of final degree of saturation are lower than actually existed during shear as they are measured after the sample has expanded in response to the release of the confining and back pressures. This reduction in degree of saturation is often mitigated by an inflow of water from the porous stones, however, Olson (1963c) reported that the volume of inflow is not significant for soils of low permeability. On the basis of corrections obtained from the application of Boyle's Law a measured degree of saturation of at least 97.5 percent is believed to represent an as-sheared degree of saturation of well over 99 percent. The Boyle's Law correction will be discussed fully in the following paragraphs. The 97.5 percent dividing line is shown in Figure 22. The 15 samples which plot below this line require further investigation

Of the 15 "substandard" samples shown in Figure 22, 12 were compacted at or wet of optimum moisture content ($S_i \geq 85\%$). Air permeability measurements reported by Barden and Sides (1970) indicated general occlusion of the

voids for these general compaction conditions. The occluded air bubbles cannot be removed by the flushing portion of the saturation procedure and must be highly compressed by the applied back pressure to achieve high degrees of saturation. Upon release of the confining and back pressures following shear these bubbles increase in volume due to direct expansion and accretion of the air leaving solution. Boyle's Law can be used to model the expansion portion of the volume change, allowing a rough estimate of the as-sheared degree of saturation to be calculated (equation 3-1).

$$p_{as} V_{as} = p_{ps} V_{ps}^* \quad (3-1)$$

p_{as} = As-sheared pore air pressure (assumed equal to the applied back pressure)

V_{as} = As-sheared pore air volume

p_{ps} = Post-shear pore air pressure (assumed atmospheric following release of the confining pressure)

V_{ps} = Post-shear pore air volume (obtained directly from the measured degree of saturation)

The actual value of p_{ps} probably lies between one atmosphere and the applied back pressure and decays with time, however it is very likely that it is significantly lowered by the release of the confining and back pressures. An illustration of the Boyle's Law "correction" is provided in the following example calculation utilizing the data for sample MW2(138). The as-sheared pore air pressure (p_{as})

* Equation (3-1) assumes constant temperature conditions.

equals 621 kN/m^2 and the post-shear pore air volume (V_{ps}) equals 839 mm^3 . The as-sheared pore air volume (V_{as}) equals $(101 \text{ kN/m}^2 \times 839 \text{ mm}^3)/621 \text{ kN/m}^2$ or 136 mm^3 . This represents an as-sheared degree of saturation of 99.5 percent. Figure 23 shows the "corrected" degrees of saturation for the 15 "substandard" samples noted in Figure 22. All but two samples exceed 99 percent saturation and all of them exceed 98.5 percent saturation. Although the accuracy of this correction is not high, the general trend is very significant and indicates that the final degrees of saturation attained in this study are generally satisfactory.

The three "substandard" samples with initial degrees of saturation of 70 percent evinced poor saturation results beyond that attributable to the expansion of trapped air upon pressure release. The Boyle's Law "correction" shows that the samples were probably adequately saturated, but the B parameter values (listed in Table 4) indicate that air was trapped elsewhere in the pore pressure measuring system. The B parameter values for these samples range from 0.70 to 0.85, whereas tests conducted later on samples compacted to identical conditions produced B parameter values from 0.92 to 1.0. The three "substandard" samples were tested prior to the complete maturation of the saturation procedure as described in section 2-6. The early saturation procedure did not incorporate the flushing process and the air trapped during placement of the rubber membrane

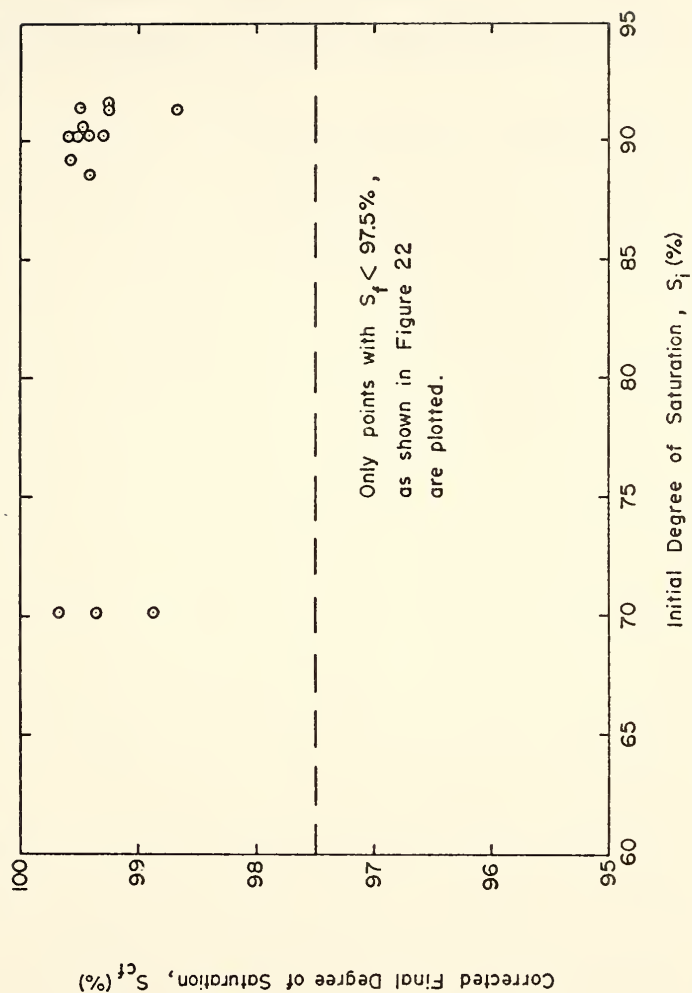


FIGURE 23 COMPARISON OF INITIAL AND CORRECTED FINAL VALUES OF DEGREE OF SATURATION

around the sample could not be removed. However, the change in pore pressure versus strain plots, shown in Figures 31 and 32, indicate no great abnormality of behavior. On this basis the results of these three tests were retained and used in later analyses.

Figure 24 shows a comparison between the experimental back pressures (Δu_E) used to attain the measured final saturation levels and the theoretical back pressures (Δu_T) required to attain the same levels according to equation (1-2) (Lowe and Johnson, 1960). The large deviation from theory for high initial degrees of saturation can be attributed to the expansion of trapped air following release of the confining and back pressures. The ratios of experimental to theoretical back pressure greater than 3 in Figure 24 are plotted in Figure 25 after recalculating them based on the theoretical back pressures required to attain the final "corrected" saturation levels. The results are now in general agreement with theory. Figure 24 indicates that theory tends to overestimate the back pressure requirements of samples with low initial degrees of saturation. However, these calculations do not take into account the large volumes of air removed from these relatively open-structured samples during the flushing process prior to back pressure application. If the increase in saturation prior to back pressure application could be determined, the theoretical back

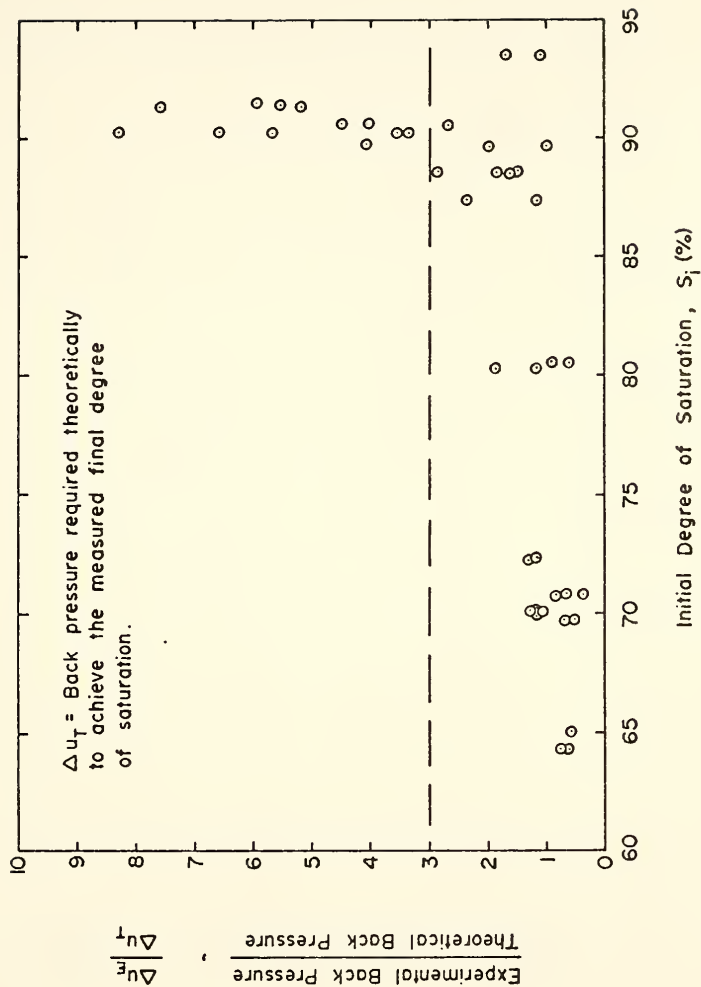


FIGURE 24 COMPARISON OF EXPERIMENTAL AND THEORETICAL BACK PRESSURES

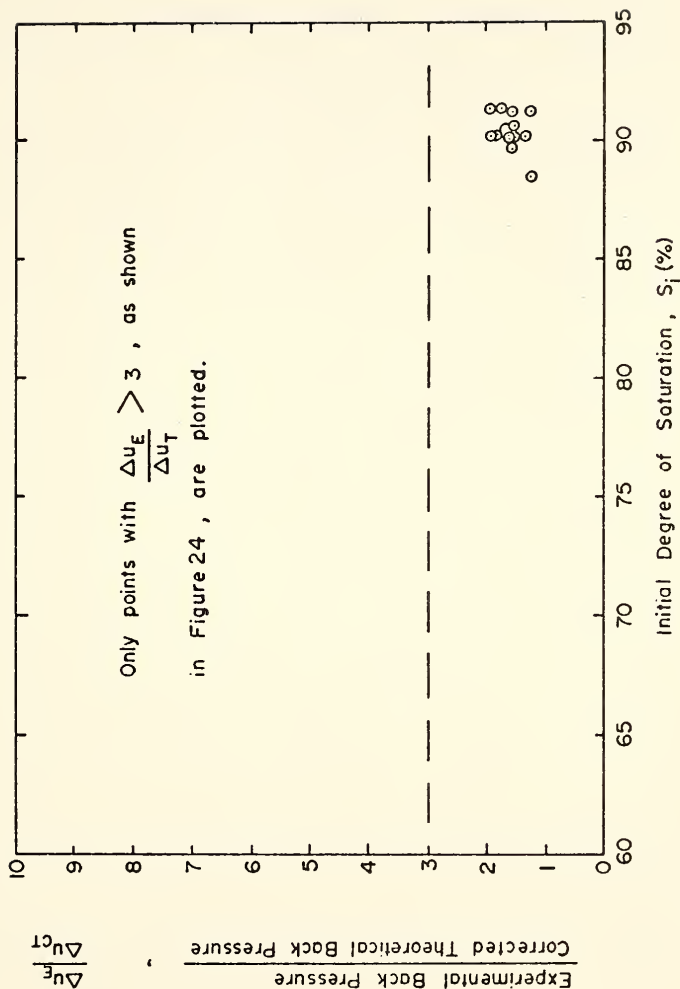


FIGURE 25 COMPARISON OF EXPERIMENTAL AND CORRECTED THEORETICAL BACK PRESSURES

pressure would be significantly reduced and closer agreement to theory would result.

3-3 Volume Change Due to Saturation and Consolidation

Long term conditions within a compacted embankment include changes in void ratio from the as-compacted condition in response to wetting and the load imposed by succeeding lifts. The mechanisms by which the volume changes occur are consolidation, swell and collapse. For this study, long term conditions were simulated in the laboratory by back pressure saturation of the compacted samples prior to consolidation under an isotropic state of stress. As noted in subsection 1-4.1, this time sequence is probably the reverse of that which occurs in situ and can result in significantly different equilibrium void ratios for compaction conditions susceptible to collapse. DiBernardo (1979) reported the results of oedometer tests on compacted Saint Croix clay which indicated collapse prone soil fabrics for samples compacted dry of optimum at standard and low energy Proctor dry densities. The possible differences in equilibrium void ratio for these conditions, unknown in magnitude, should be kept in mind when analyzing the data.

The volume changes which occur in response to wetting and loading are very important in terms of the successful long term performance of the compacted embankment. First, the settlement of the embankment due to internal

compression must lie within the limits imposed by the design assumptions and acceptability criteria. Second, the strength of the soil at a given confining pressure is greatly altered by changes in void ratio and must be modified for use in stability analyses.

Figures 26, 27 and 28 show the range of saturated-consolidated conditions obtained from each as-compacted condition investigated in this study. The spread of saturated-consolidated dry density values (1550 to 1700 kg/m³) is significantly smaller than the spread for the same samples in the as-compacted state (1525 to 1825 kg/m³). All samples compacted to modified Proctor dry densities evince net swell following saturation and consolidation. Samples compacted to standard Proctor dry densities evince either net swell or net compression, depending upon the consolidation pressure used, and samples compacted to low energy Proctor dry densities evince net compression. The equilibrium percent volume change due to saturation and consolidation ($\Delta V/V_o$, %) is shown for each sample in Table 5. ($\Delta V/V_o$ (%) = $\frac{\Delta e}{1 + e_o} \times 100$, where Δe = the change in void ratio from the as-compacted to the saturated-consolidated condition, and e_o = the initial void ratio.) The contours of equal percent volume change in Figure 29 further illustrate the effects of changes in initial void ratio (e_o , or dry density) and consolidation pressure (σ'_c). As void ratio increases (dry density decreases) for a given consolidation pressure, the percent

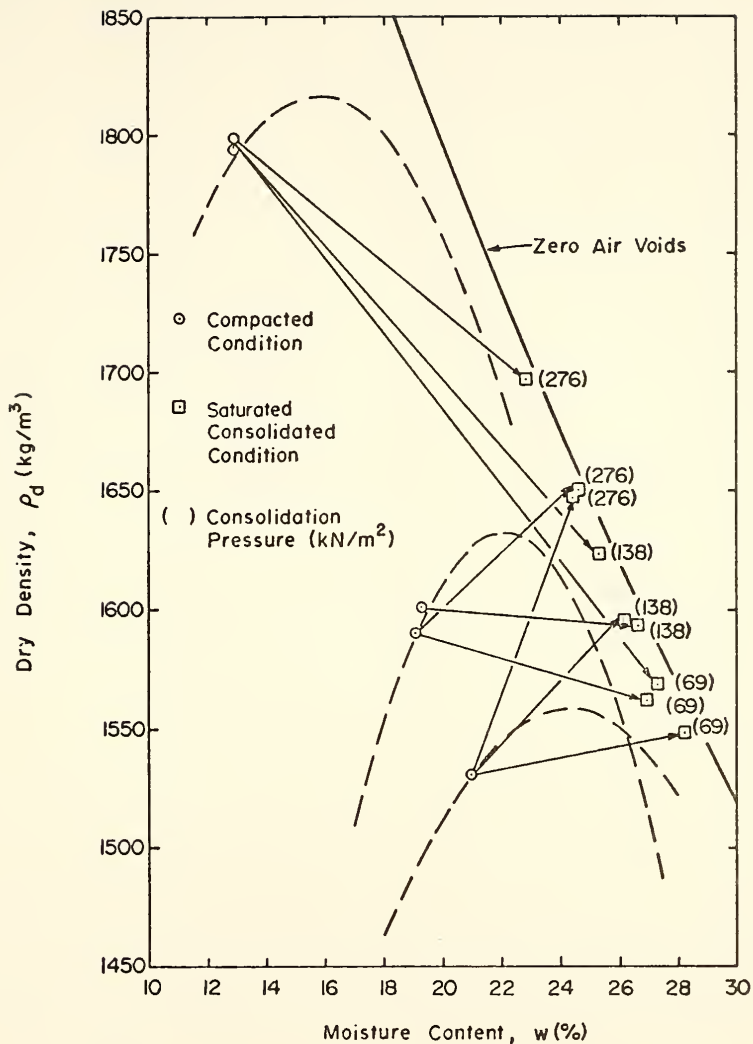


FIGURE 26

MOISTURE CONTENT AND DRY DENSITY VALUES BEFORE AND AFTER SATURATION AND CONSOLIDATION (COMPACTION DRY OF OPTIMUM MOISTURE CONTENT)

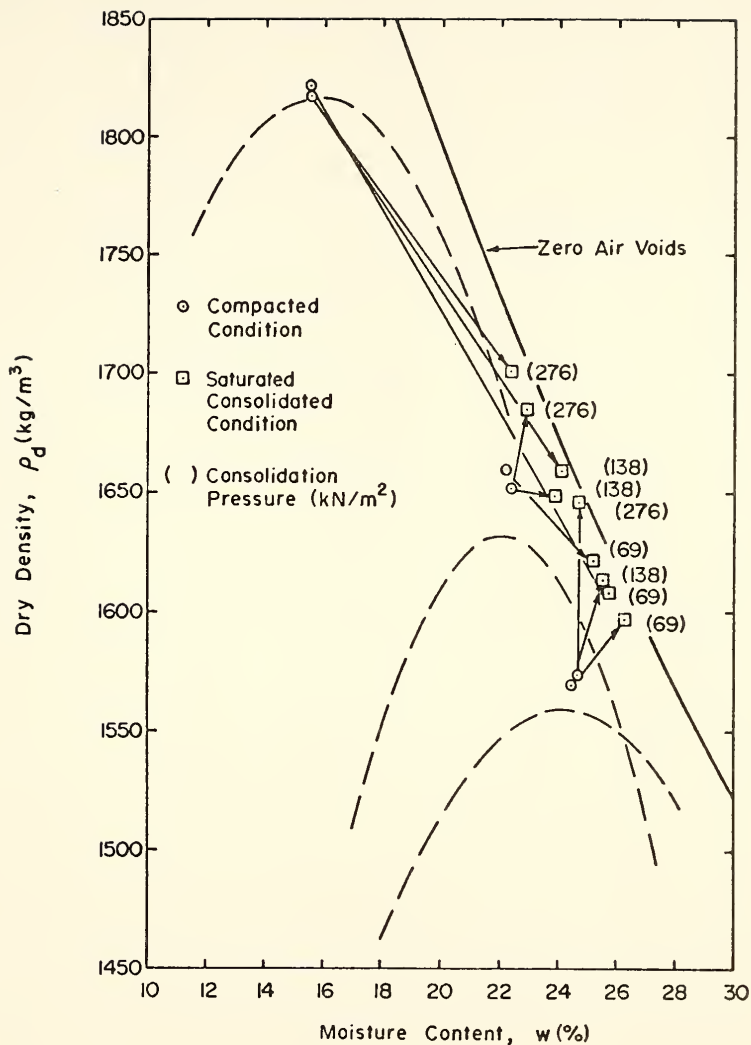


FIGURE 27

MOISTURE CONTENT AND DRY DENSITY VALUES BEFORE AND AFTER SATURATION AND CONSOLIDATION (COMPACTION AT OPTIMUM MOISTURE CONTENT)

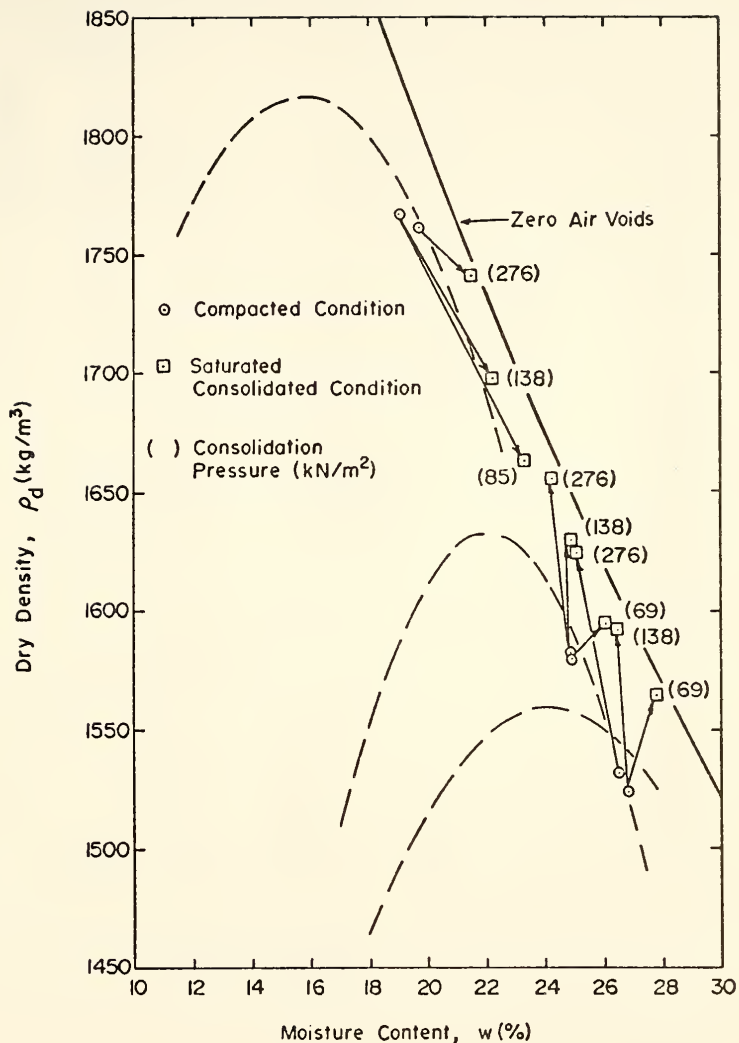


FIGURE 28 MOISTURE CONTENT AND DRY DENSITY VALUES BEFORE AND AFTER SATURATION AND CONSOLIDATION (COMPACTION WET OF OPTIMUM MOISTURE CONTENT)

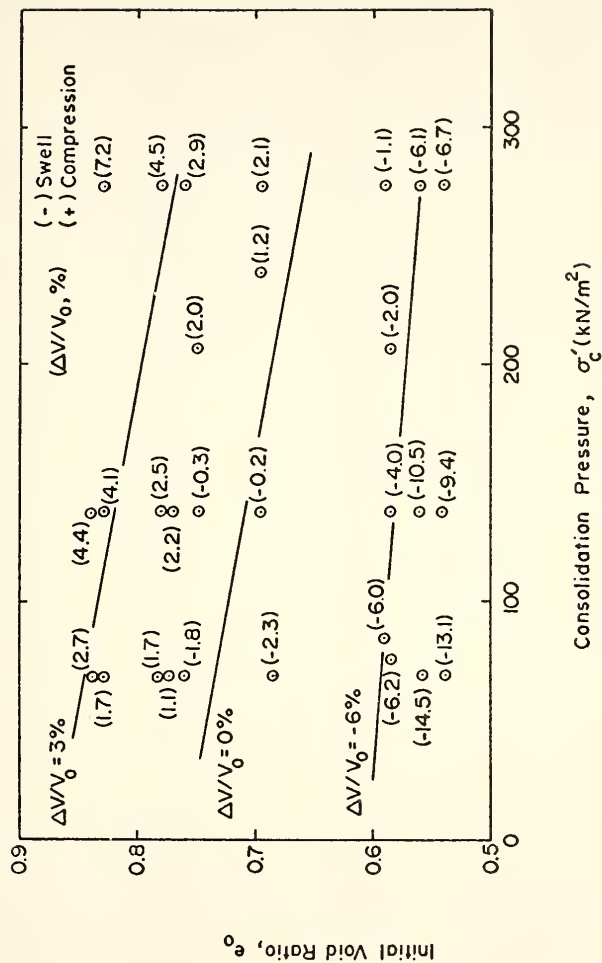


FIGURE 29

CONTOURS OF PERCENT VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION

TABLE 5. VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION

Sample No.	Initial Void Ratio e_o	Final Void Ratio e_f	Percent Volume Change Due To Saturation And Consolidation, $\Delta V/V_o$ (%) (+ = Compression)
MD1 (69)	0.558	0.784	-14.5
MD2 (138)	0.560	0.724	-10.5
MD3 (276)	0.560	0.655	- 6.1
MD4 (276)	0.558	0.649	- 5.8
MO1 (69)	0.538	0.740	-13.1
MO2 (138)	0.541	0.686	- 9.4
MO3 (276)	0.541	0.644	- 6.7
MO4 (69)	0.538	0.748	-13.7
MW1 (85)	0.590	0.685	- 6.0
MW2 (138)	0.585	0.648	- 4.0
MW3 (276)	0.584	0.603	- 1.2
MW4 (76)	0.585	0.683	- 6.2
MW5 (207)	0.584	0.615	- 2.0
MW6 (276)	0.590	0.607	- 1.1
SD1 (69)	0.760	0.791	- 1.8
SD2 (138)	0.748	0.754	- 0.3
SD3 (276)	0.764	0.696	3.9
SD4 (138)	0.764	0.756	0.5
SD5 (207)	0.748	0.713	2.0
SD6 (276)	0.760	0.709	2.9
SD7 (138)	0.760	0.763	- 0.2
SO1 (69)	0.686	0.725	- 2.3
SO2 (138)	0.695	0.698	- 0.2
SO3 (276)	0.695	0.660	2.1
SO4 (239)	0.695	0.675	1.2
SW1 (69)	0.773	0.754	1.1
SW2 (138)	0.773	0.717	3.2
SW3 (276)	0.770	0.690	4.5
SW4 (138)	0.770	0.732	2.2
LD1 (69)	0.828	0.807	1.2
LD2 (138)	0.828	0.754	4.1
LD3 (276)	0.828	0.697	7.2
LD4 (69)	0.828	0.797	1.7
LO1 (69)	0.783	0.752	1.7
LO2 (138)	0.779	0.734	2.5
LO3 (276)	0.779	0.699	4.5
LO4 (69)	0.779	0.771	0.5
LO5 (138)	0.783	0.738	2.5
LO6 (276)	0.779	0.685	5.3
LW1 (69)	0.838	0.788	2.7
LW2 (138)	0.838	0.757	4.4
LW3 (276)	0.828	0.723	5.7

volume change due to saturation and consolidation increases (less net swell or greater net compression). Holtz and Gibbs (1956) reported this same trend but warned that continually increasing dry density at a constant moisture content will eventually result in an increase in $\Delta V/V_o$ due to the influence of degree of saturation on swell. As expected, increases in consolidation pressure result in higher values of percent volume change (less net swell or greater net compression). Compaction moisture content also affects percent volume change due to saturation and consolidation, but trends are more difficult to isolate. Holtz and Gibbs (1956) report for swell tests on compacted clay that increasing the compaction moisture content for a given dry density results in decreased percent swell. Although the same trend is observed in this study, it is no longer visible after consolidation for those samples compacted to standard Proctor and low energy Proctor dry densities. For these cases, the samples compacted dry of optimum exhibit percent volume changes either equal to or greater than (less net swell or more net compression) those found for compaction wet of optimum. This behavior can be partially explained by the slightly steeper slopes of the final void ratio versus logarithm of consolidation pressure plots, shown in Figure 30, for samples compacted dry of optimum moisture content. However, the fact that samples compacted dry of optimum to modified Proctor dry densities evince

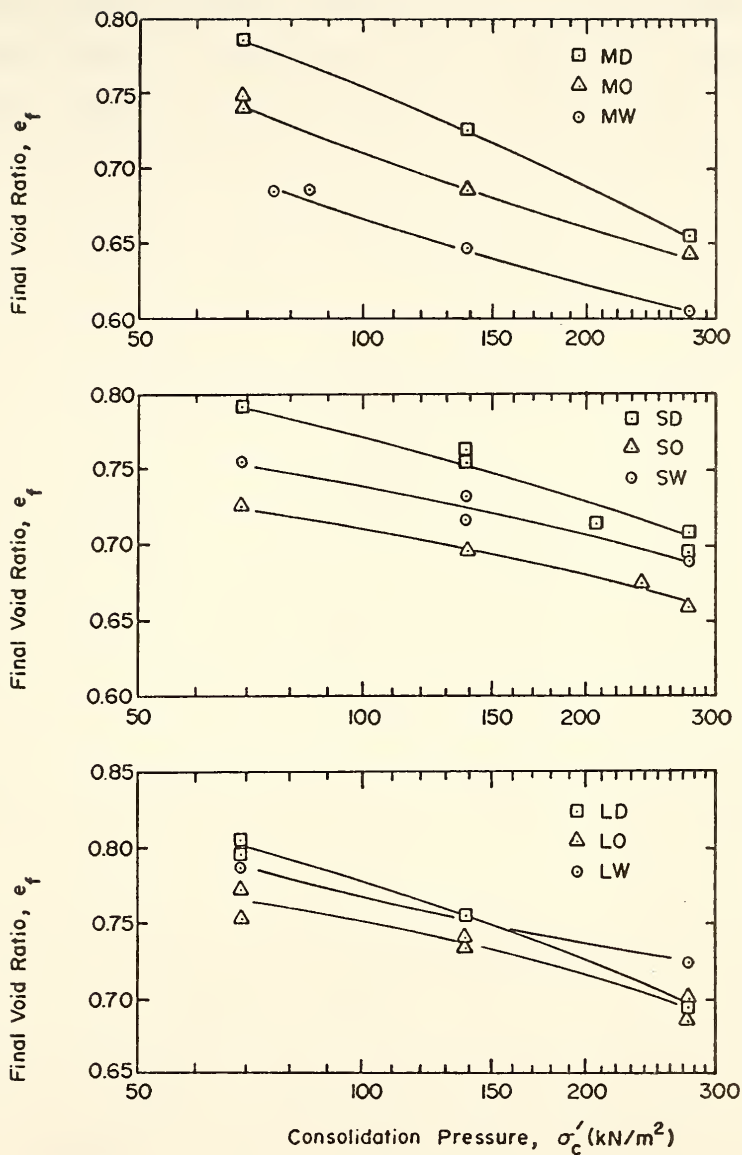


FIGURE 30

FINAL VOID RATIO VERSUS LOGARITHM
OF CONSOLIDATION PRESSURE

greater net swell than those compacted wet of optimum shows that other factors are involved as well. DiBernardo (1979) showed that samples of Saint Croix clay compacted dry of optimum to standard Proctor and low energy Proctor dry densities are susceptible to collapse upon wetting. Mishu (1963) presented results of tests on two clays compacted to collapse prone conditions which indicated that saturation prior to consolidation results in greater volumetric reduction than obtained through collapse due to saturation after consolidation. As the former time sequence was utilized in the testing procedure for this study, it is possible that the percent volume change values for the collapse prone dry side compaction samples are higher than would otherwise be expected. For compaction conditions not susceptible to collapse an increase in compaction moisture content appears to result in an increase in percent volume change due to saturation and consolidation (less net swell or more net compression).

3-4 Stress-Strain and Pore Water

Pressure Response

The stress versus strain and pore water pressure versus strain curves for samples compacted to standard Proctor dry densities, dry of, at and wet of optimum are shown in Figures 31, 32 33 and 34. The curves for the remainder of the compaction conditions are shown in Figures D1 to D8. All of the stress versus strain curves classify

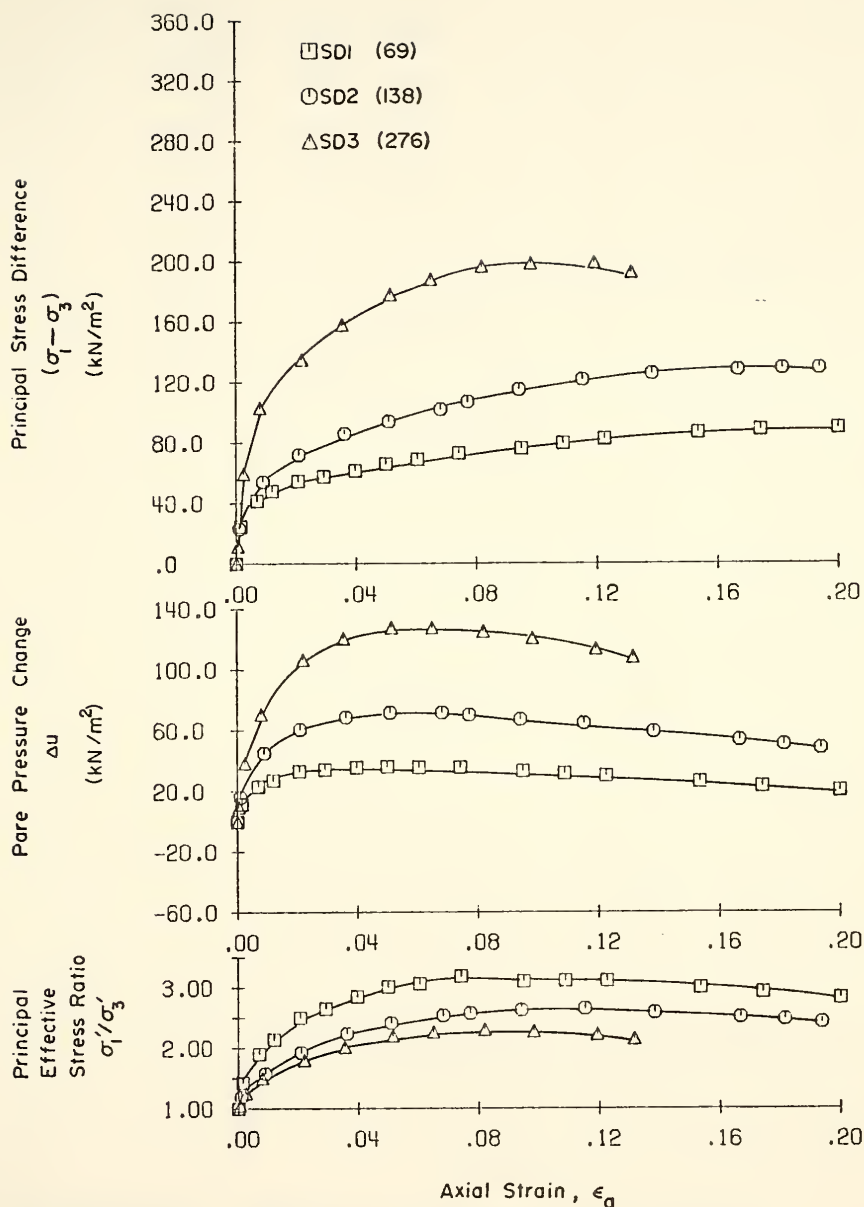


FIGURE 31 CIU RESULTS, TESTS SD1 TO SD3

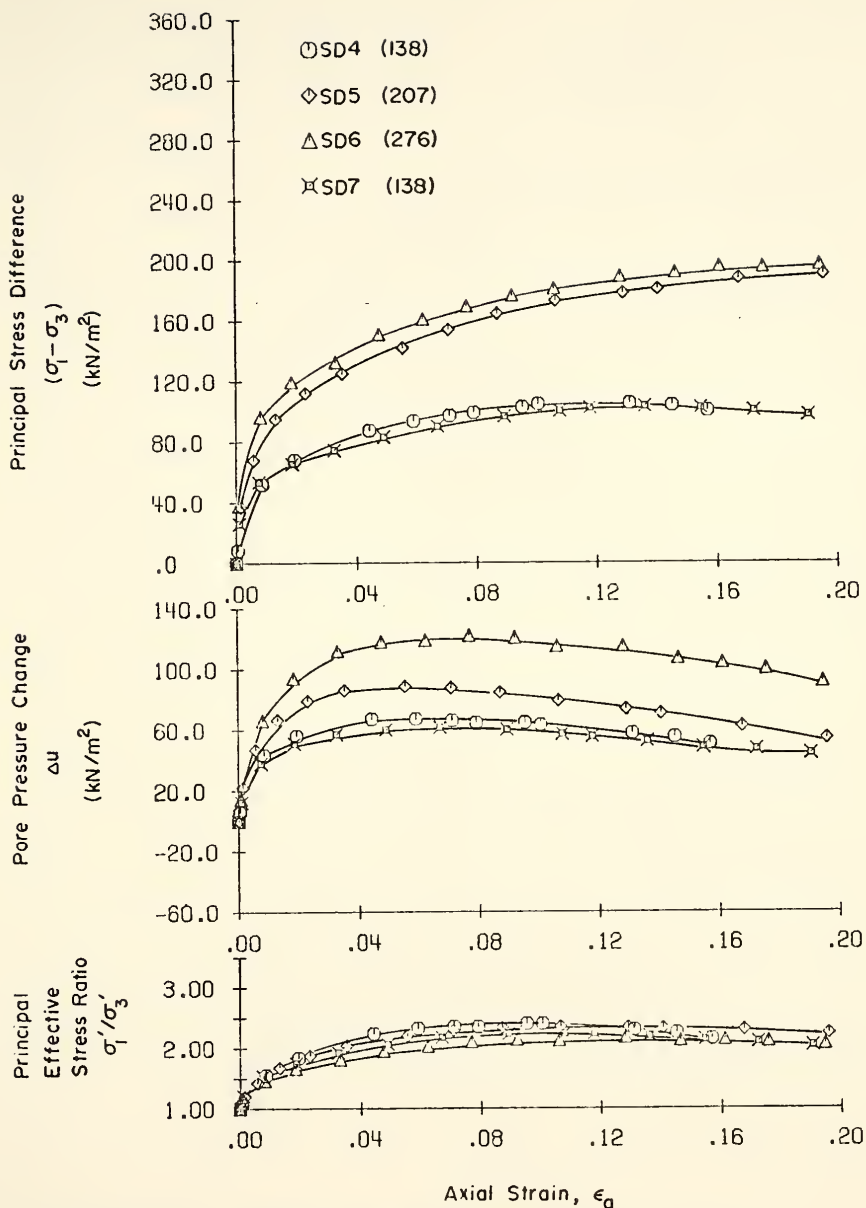


FIGURE 32 CIU RESULTS, TESTS SD4 TO SD7

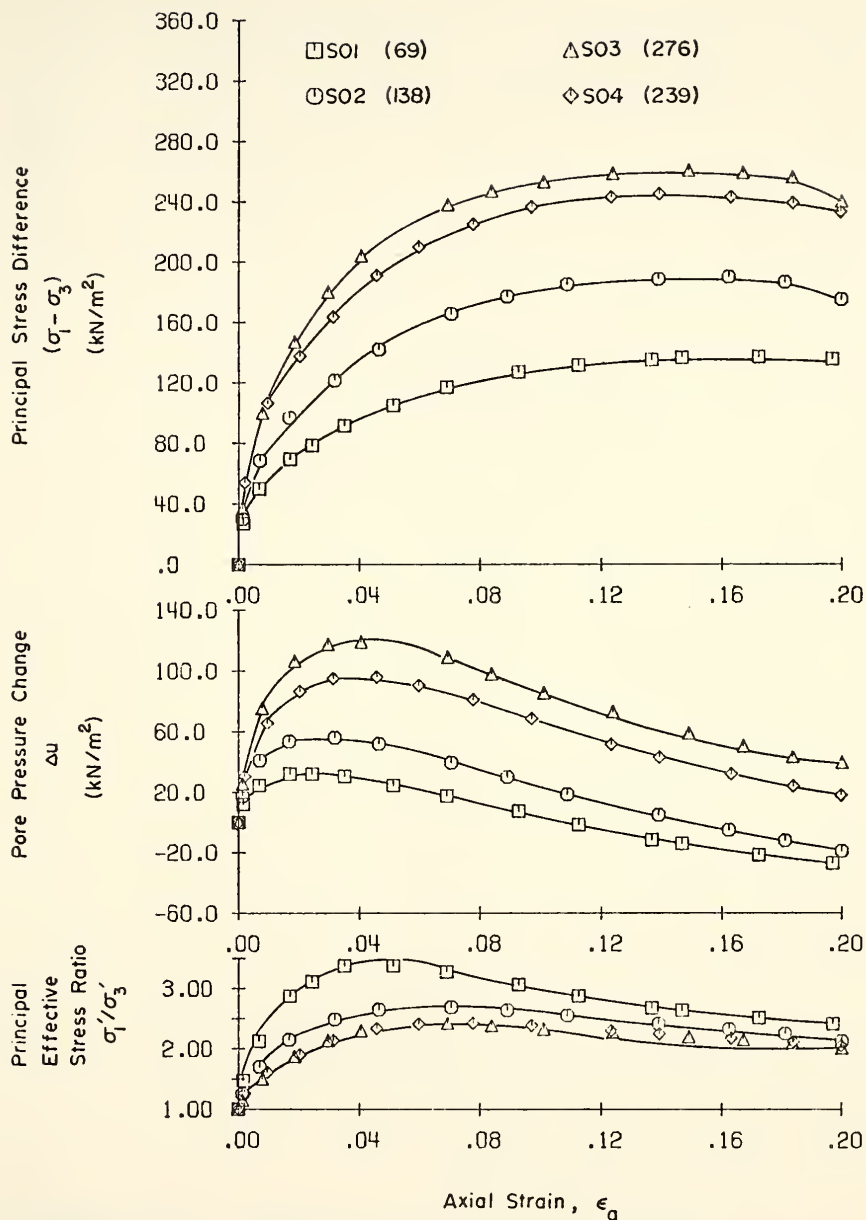


FIGURE 33 CIU RESULTS, TESTS S01 TO S04

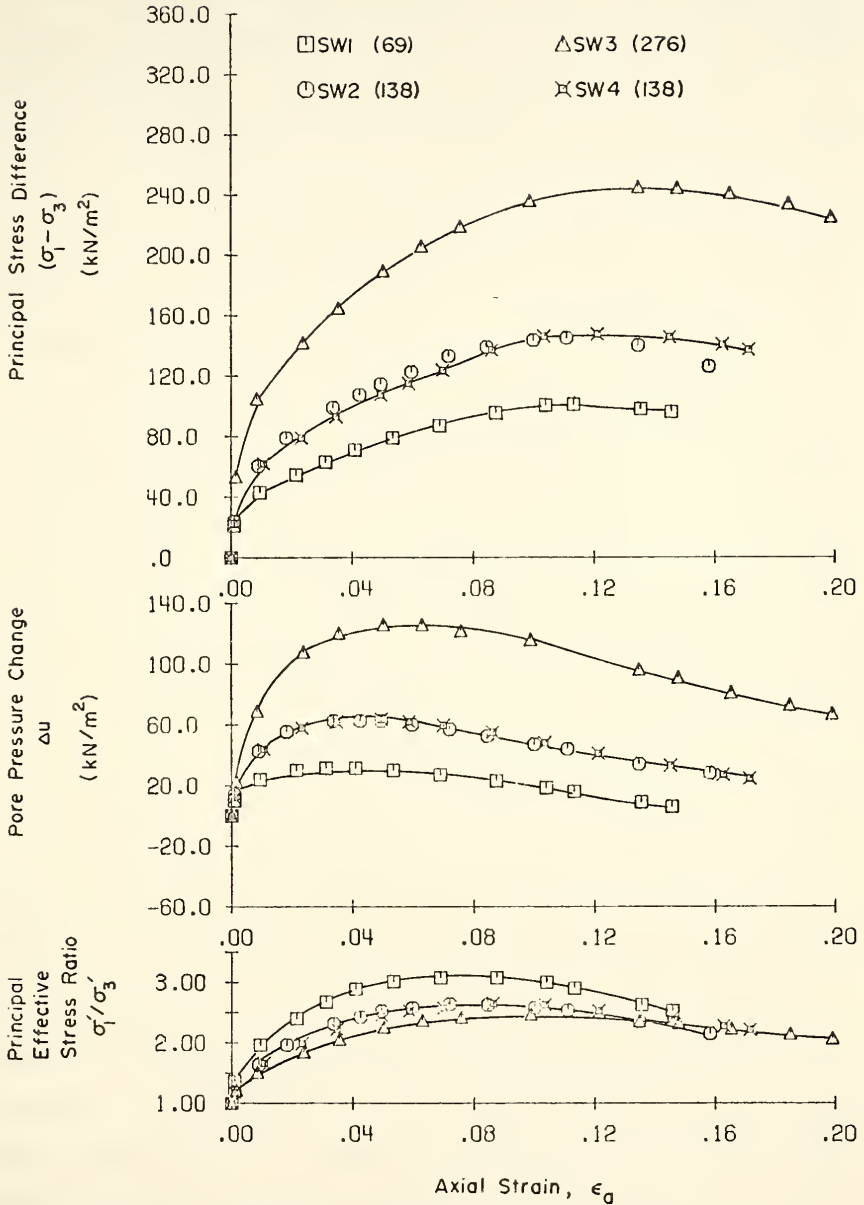


FIGURE 34 CIU RESULTS, TESTS SW1 TO SW4

as either Type I or Type II, as defined in the system presented by Casagrande and Hirschfeld (1962) and discussed in subsection 1-4.3. All further discussion of "types" of stress versus strain curves will be in terms of this system. Type I and Type II curves indicate well-rounded curves with the possible exception of an initial straight line section which terminates before half the maximum shear stress is attained.

Seed and Chan (1959) presented the results of an investigation in which samples were compacted dry and wet of optimum moisture content and later saturated and consolidated to common values of dry density, moisture content and consolidation pressure prior to undrained shearing. They concluded that these samples would fail at approximately the same values of shear stress and pore water pressure although the shapes of the stress versus strain and pore pressure versus strain curves could be significantly different for the two compaction conditions. Figure 6 showed the curves obtained by shearing samples of silty clay compacted dry and wet of optimum. Compaction dry of optimum produced a Type III stress versus strain curve while compaction wet of optimum produced a Type V curve. Seed and Chan (1959) also presented the results of tests conducted on other soil types which showed little difference between stress versus strain curves for the different compaction conditions, much like Saint Croix clay. It appears that changes in fabric during

the swelling process may be responsible for the lack of difference between curves, as all of the other soils, including Saint Croix clay, have greater swell potential than the silty clay.

The deviator stress at failure $(\sigma_1 - \sigma_3)_f$ for each of the samples tested in this study is plotted versus the measured final void ratio (e_f) in Figure 35. As noted by Seed and Chan (1959), samples compacted to different initial conditions and saturated and consolidated to common values of dry density, moisture content and consolidation pressure, fail at approximately the same deviator stress. Pore water pressures at failure, plotted in Figure 36, are also approximately the same under these conditions. The compacted silty clay tested by Seed and Chan (1959) showed a total variation of 20 kN/m^2 for both deviator stress and pore water pressure at failure for samples tested at a common final condition. For the compacted Saint Croix Clay utilized in this study, total variation of the deviator stress at failure was 40 kN/m^2 and total variation of the pore water pressure at failure was 50 kN/m^2 .

Much of the variation observed in the study presented by Seed and Chan (1959) and part of the variation observed in this study can be attributed to differences between the measured values of final void ratio and the values which actually existed on the failure plane at failure. In this study and the study by Seed and Chan (1959), the void

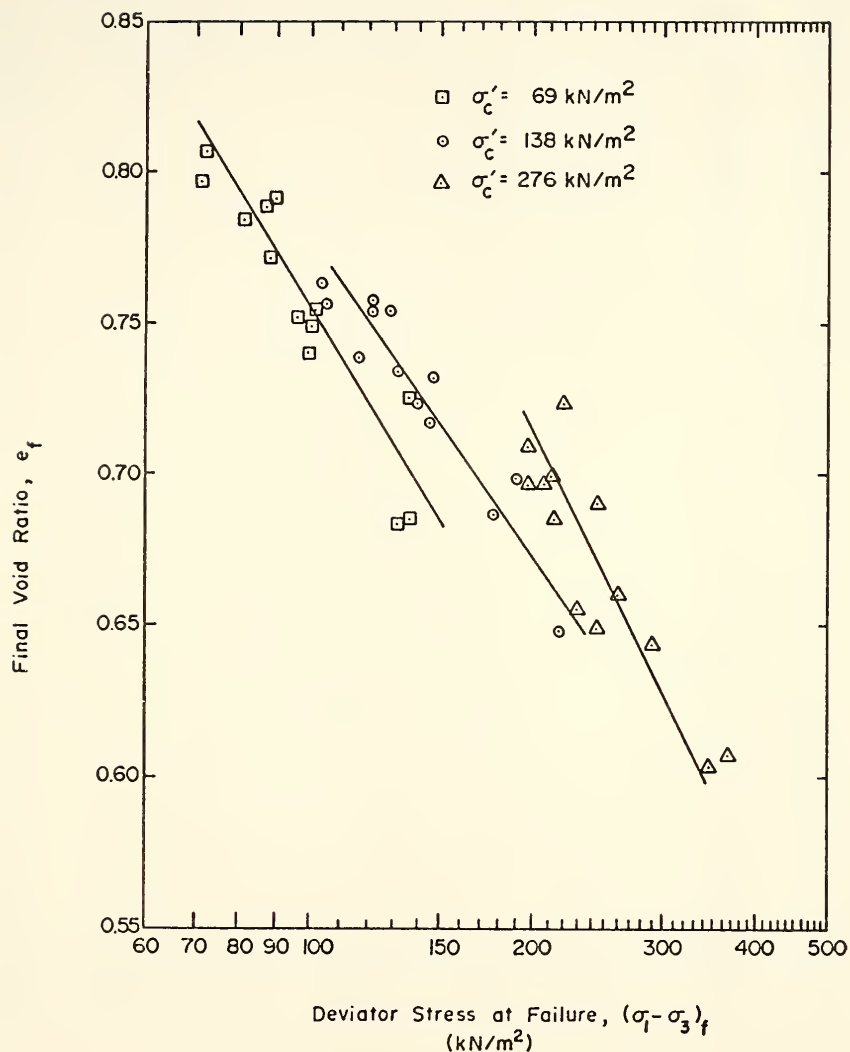


FIGURE 35 RELATIONSHIP BETWEEN DEVIATOR STRESS AT FAILURE AND FINAL VOID RATIO

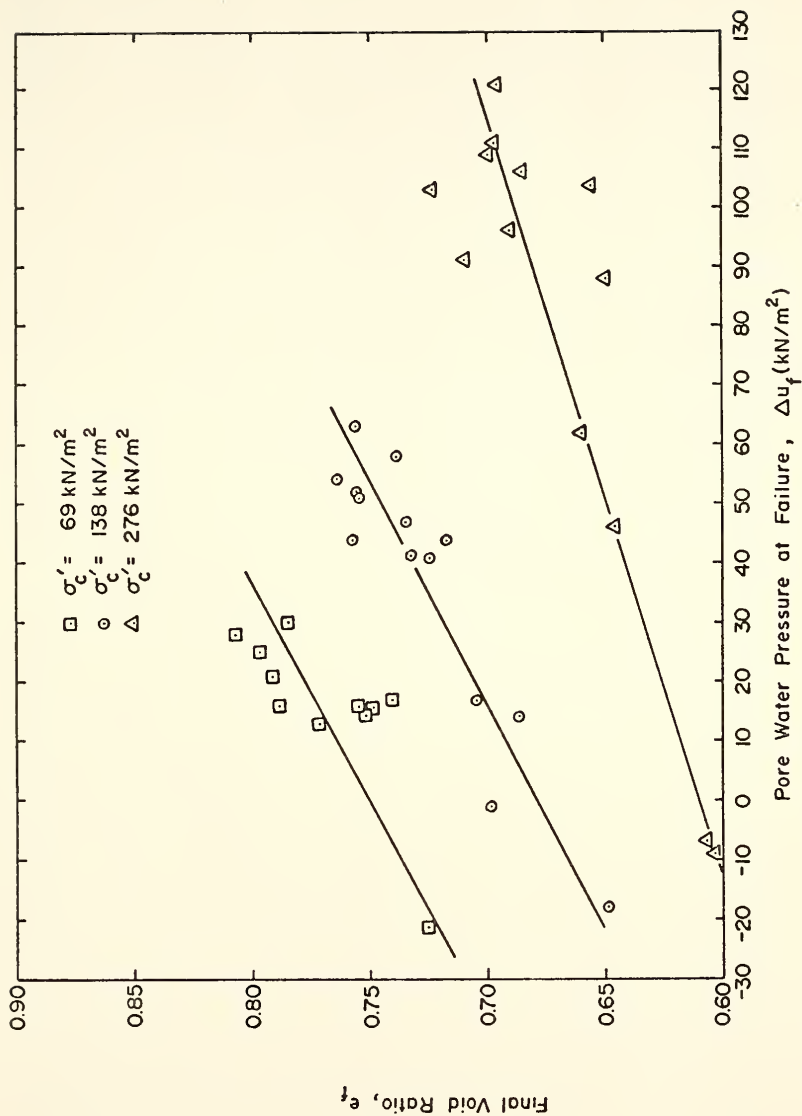


FIGURE 36 RELATIONSHIP BETWEEN PORE PRESSURE AT FAILURE AND FINAL VOID RATIO

ratio of the whole triaxial sample was measured and assumed equal to the void ratio on the failure plane at failure. However, as noted in subsection 1-2.3, dissipation of pore pressure gradients during undrained testing is accomplished by moisture migration. In a saturated system, moisture migration implies void ratio changes. It is therefore highly unlikely that the void ratio at failure on the failure plane is equal to the average value obtained from measurements on the whole sample. In addition, the procedure utilized in this study to measure the final void ratio results in a slightly high value of void ratio due to expansion of the sample following release of the confining and back pressures.

Variation in pore water temperature during undrained shear is most likely responsible for much of the scatter observed in this study in excess of that noted by Seed and Chan (1959). The shear tests for the Seed and Chan study were all conducted during the morning hours when temperature changes were small and consistent (Seed, Mitchell and Chan, 1960). The samples for this study were sheared as soon as ready, and were approximately evenly distributed over morning, afternoon and evening periods. Water temperature measurements in the laboratory indicated that a change of $\pm 1^{\circ}$ centigrade could be expected during the 3 hour period needed to shear a sample. Estimates of the pore pressure induced by the differential expansion and/or contraction of the soil skeleton and pore fluid per unit temperature change vary

greatly. Seed, Mitchell and Chan (1960) reported coefficients as high as 14.4 kN/m^2 per degree centigrade, while Campanella and Mitchell (1968) reported a low value of 2.5 kN/m^2 per degree centigrade. It is therefore possible for two identical tests to develop pore water pressures at failure differing by 5 to 30 kN/m^2 if one is sheared during the afternoon and the other during the evening. The pore pressure differences are also reflected in differences in deviator stresses.

Variation in pore water pressure at failure due to temperature change does not affect the value of the effective stress strength parameters, as deviator stress at failure changes in the opposite sense. The result is a different point of failure on the same failure envelope. However, such variation does have a very significant effect on the calculated value of Skempton's A parameter at failure (A_f). A good example is provided by comparison of the failure conditions of samples SD2(138) and SD4(138), which are listed along with the values for all the samples in Table 6. The drop in pore water pressure at failure and the increase in deviator stress at failure from sample SD2(138) to sample SD4(138) are within the range believed to be attributable to temperature changes during shear. The difference in the A_f values, 0.20 ($64/105 - 51/129 = 0.60 - 0.40 = 0.20$), is highly significant and indicates that scatter will be a significant factor during trend isolation for this data.

TABLE 6. FAILURE CONDITIONS FOR CONSOLIDATED UNDRAINED TRIAXIAL TESTS

Sample No.	Principal Stress Difference At Failure $(\sigma_1 - \sigma_3)_f$ (kN/m ²)	Pore Pressure Change At Failure, Δu_f (kN/m ²)	Skempton's A Parameter At Failure A_f	Final Void Ratio e_f
MD1 (69)	81	30	0.37	0.784
MD2 (138)	138	41	0.30	0.724
MD3 (276)	229	104	0.45	0.655
MD4 (276)	244	88	0.36	0.649
MO1 (69)	99	17	0.17	0.740
MO2 (138)	177	14	0.08	0.686
MO3 (276)	290	46	0.16	0.644
MO4 (69)	100	16	0.16	0.748
MW1 (85)	137	15	0.11	0.685
MW2 (138)	218	- 18	-0.08	0.648
MW3 (276)	348	- 9	-0.03	0.603
MW4 (76)	132	0	0.00	0.683
MW5 (207)	270	17	0.06	0.615
MW6 (276)	368	- 7	-0.02	0.607
SD1 (69)	89	21	0.24	0.791
SD2 (138)	129	51	0.40	0.754
SD3 (276)	199	121	0.61	0.696
SD4 (138)	105	63	0.60	0.756
SD5 (207)	190	54	0.28	0.713
SD6 (276)	197	91	0.46	0.709
SD7 (138)	103	54	0.52	0.763
SO1 (69)	137	- 21	-0.15	0.725
SO2 (138)	190	- 1	-0.01	0.698
SO3 (276)	260	62	0.24	0.660
SO4 (239)	245	43	0.18	0.675
SW1 (69)	101	16	0.16	0.754
SW2 (138)	145	44	0.30	0.717
SW3 (276)	245	96	0.39	0.690
SW4 (138)	147	41	0.28	0.732
LD1 (69)	72	28	0.39	0.807
LD2 (138)	122	51	0.42	0.754
LD3 (276)	208	111	0.53	0.697
LD4 (69)	71	25	0.35	0.797
LO1 (69)	96	14	0.15	0.752
LO2 (138)	132	47	0.36	0.734
LO3 (276)	211	109	0.52	0.699
LO4 (69)	88	13	0.15	0.771
LO5 (138)	117	58	0.50	0.738
LO6 (276)	213	106	0.50	0.685
LW1 (69)	87	16	0.18	0.788
LW2 (138)	121	44	0.36	0.757
LW3 (276)	220	103	0.47	0.723

Figure 37 shows that A_f increases as consolidation pressure increases for samples compacted to the same initial condition. As an increase in consolidation pressure represents a decrease in overconsolidation ratio, this trend is in agreement with the results presented by Henkel (1956) for remolded Weald and London clays. Henkel's results were shown in Figure 8. The trend, very strong for samples compacted to low energy Proctor and standard Proctor dry densities, is not evident for samples compacted to modified Proctor dry densities. The low A_f values for these samples indicate that the soil did not behave in a normally consolidated or slightly overconsolidated manner. It is therefore very likely that the trend is weak at these compaction dry densities and is simply obscured by the scatter in the data. Figure 38 shows for constant values of initial degree of saturation and consolidation pressure that A_f increased as the compacted dry density decreased.

Abeyesekera (1978) conducted consolidated-undrained triaxial tests on back pressure saturated samples of compacted shale. He reported, for the range of compaction conditions investigated, that decreasing the compaction moisture content resulted in higher A_f values for a constant consolidation pressure. The results of this study confirmed this trend, as shown in Figure 39. Higher A_f values for a given consolidation pressure imply a lower overconsolidation ratio. It is possible that swell, which

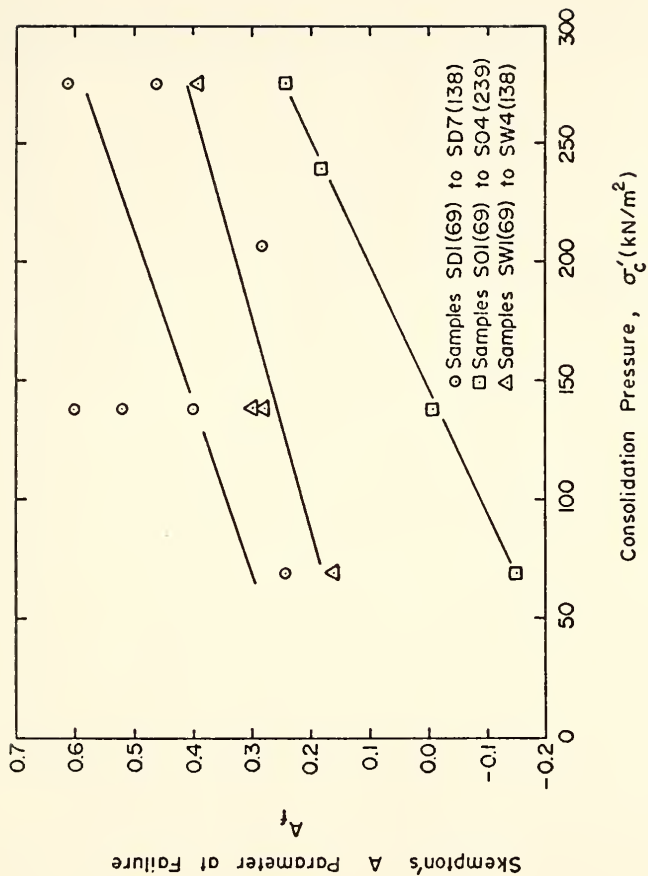


FIGURE 37 SKEMPTON'S A PARAMETER AT FAILURE VERSUS CONSOLIDATION PRESSURE

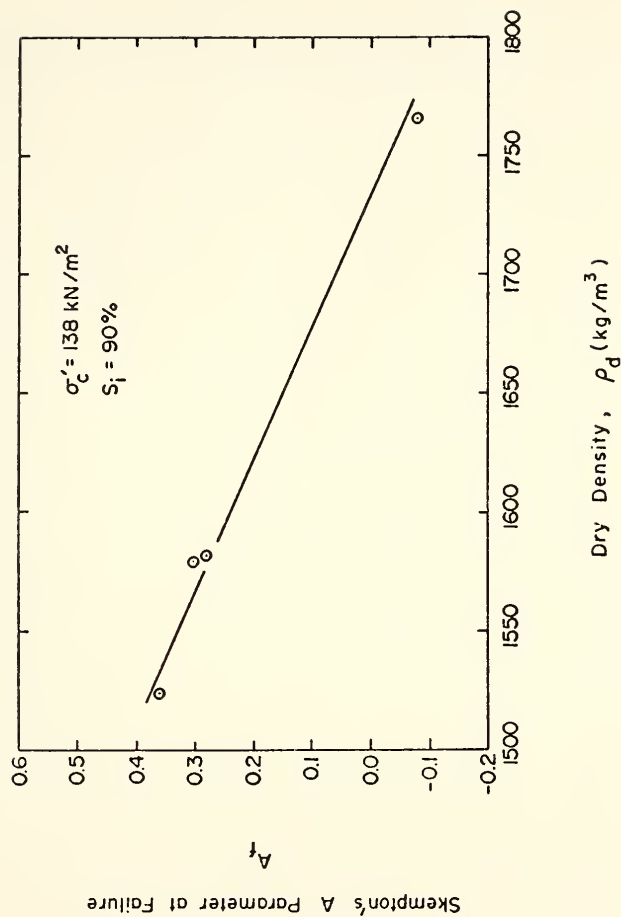


FIGURE 38

SKEMPTON'S A PARAMETER AT FAILURE VERSUS AS-COMPACTED DRY DENSITY

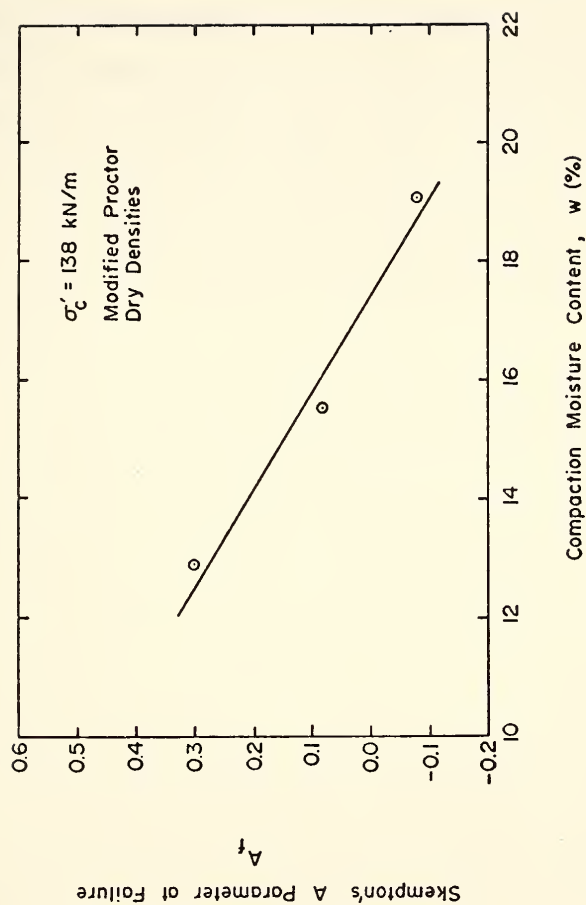


FIGURE 39 SKEMPTON'S A PARAMETER AT FAILURE VERSUS COMPACTION MOISTURE CONTENT

increases as compaction moisture content decreases, may have had a destructive effect on soil "memory". Kneading compactor foot pressure (p_c) and compactive prestress ($\hat{p}_s$) fail as representations of maximum past pressure as they cannot explain this trend. DiBernardo (1979) showed that compactive prestress increased as the compaction moisture content decreased. Kneading compactor foot pressure was also higher for compaction dry of optimum than wet of optimum.

3-5 Effective Stress Strength Parameters

The effective stress strength parameters, c' and ϕ' , were obtained from stress path plots by subjecting the values of q_f and p'_f to simple linear regression analysis ($q_f = q$ at failure = $(\sigma_1 - \sigma_3)_f / 2$ and $p'_f = p'$ at failure = $(\sigma'_1 + \sigma'_3)_f / 2$). The p'_f values were utilized as the independent variable and the q_f values as the dependent variable. Examples of the stress paths and failure envelopes are shown in Figures 40, 41, 42 and 43. The plots for the remainder of the compaction conditions are shown in Figures D9 to D16. Equation (3-2) was utilized to determine the effective stress strength parameters from the stress path plots.

$$q_f = \alpha + \beta p'_f = c' \cos \phi' + p'_f \sin \phi' \quad (3-2)$$

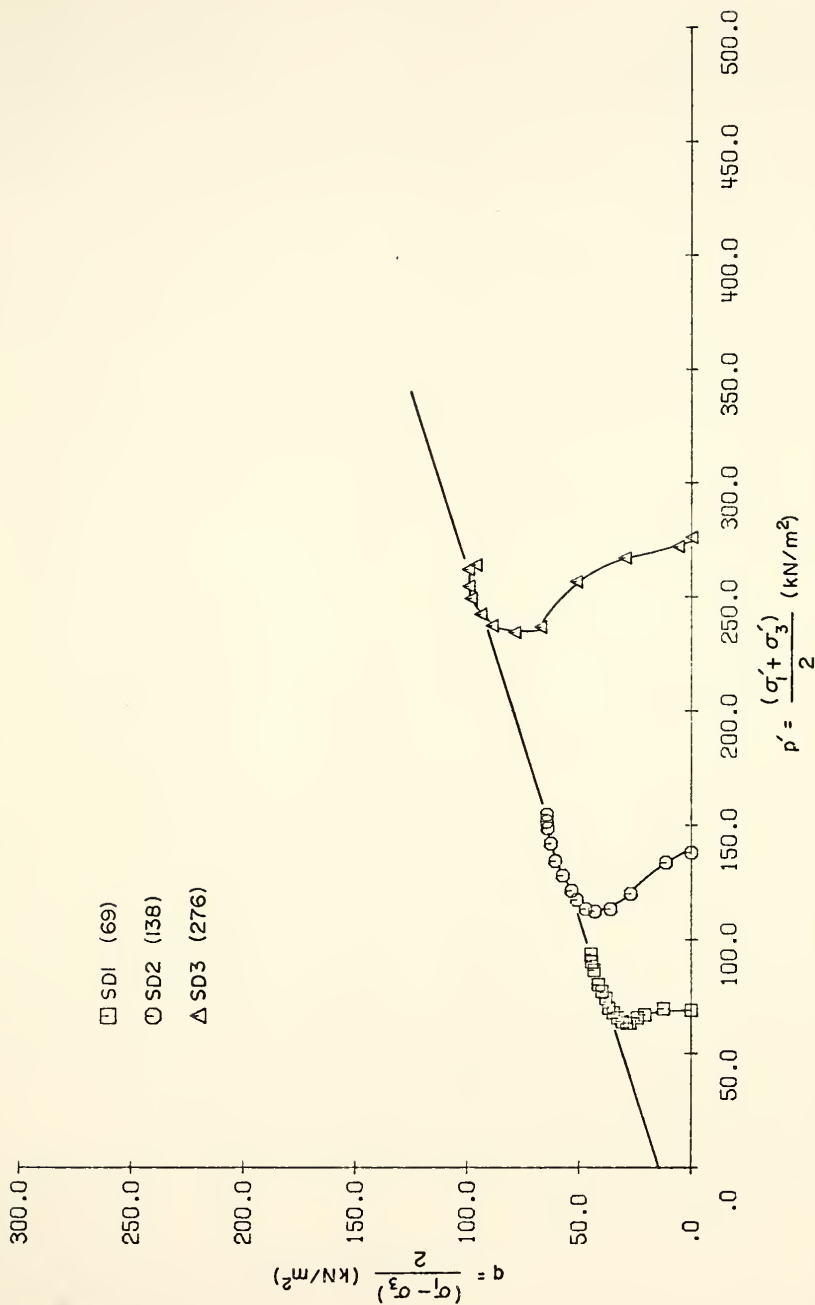


FIGURE 40 EFFECTIVE STRESS PATHS, TESTS SD1 TO SD3

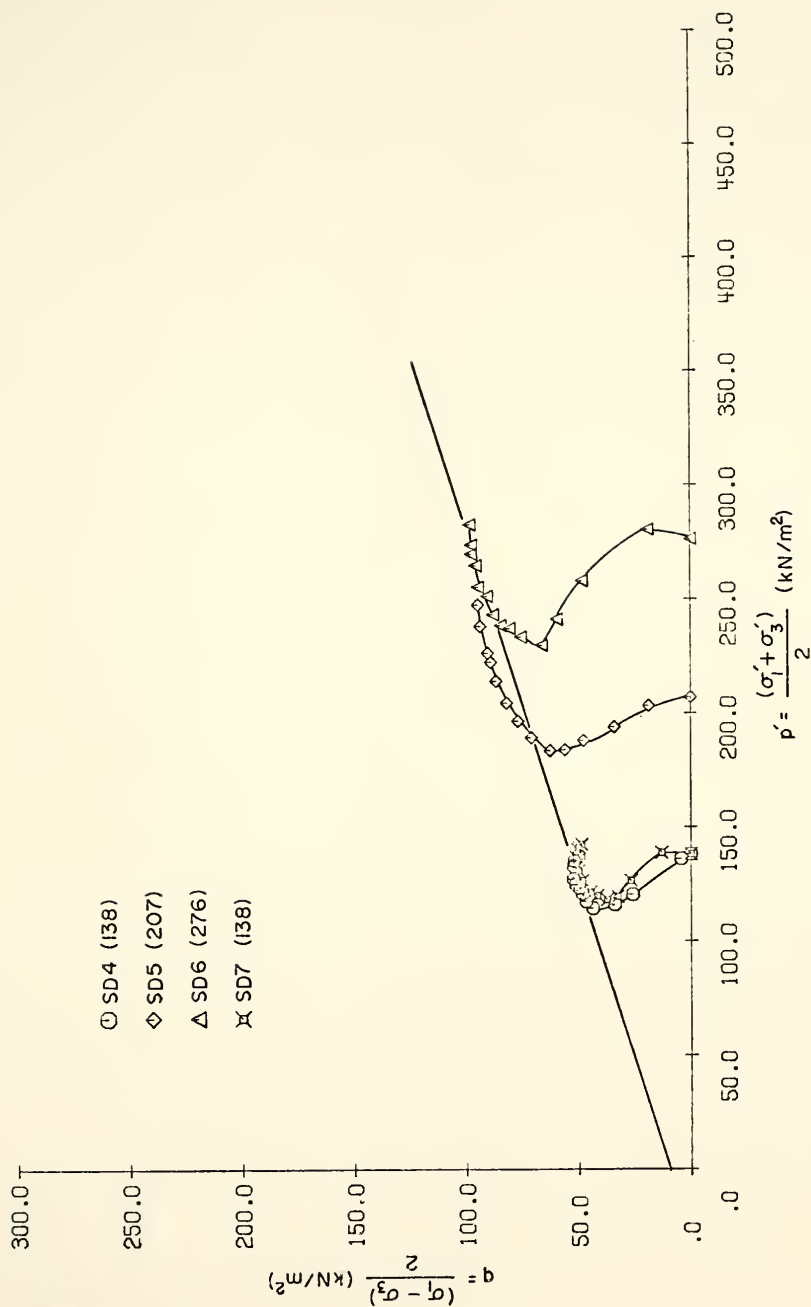


FIGURE 41 EFFECTIVE STRESS PATHS, TESTS SD4 TO SD7

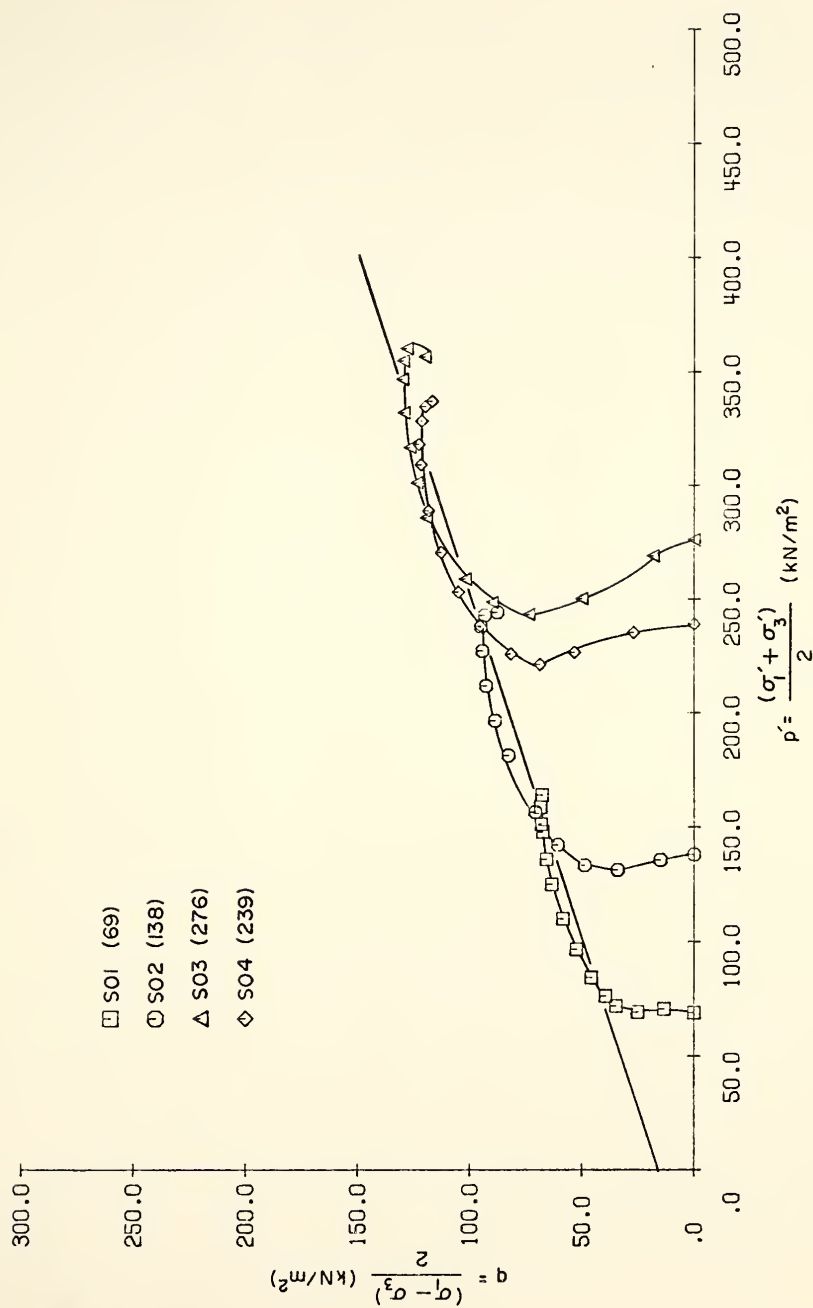


FIGURE 42 EFFECTIVE STRESS PATHS, TESTS SO1 TO SO4

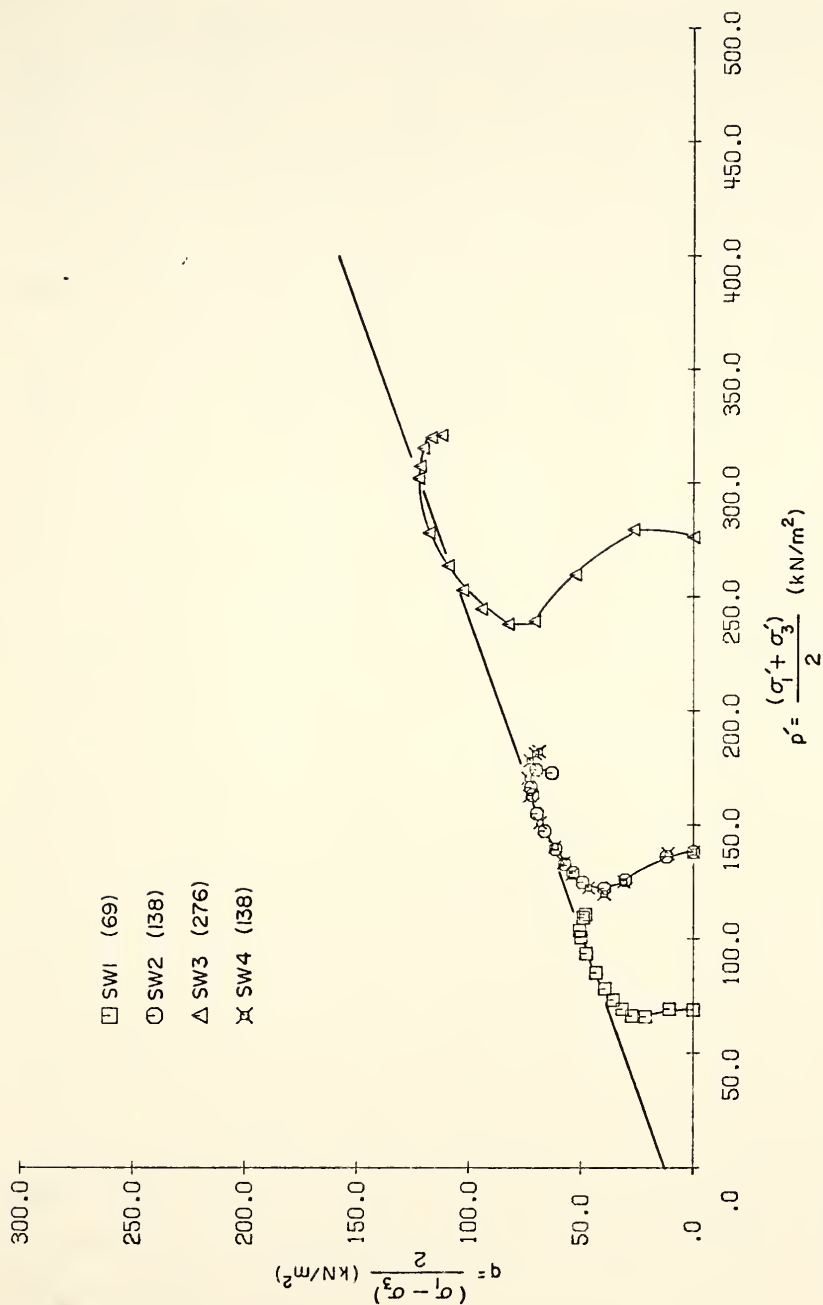


FIGURE 43 EFFECTIVE STRESS PATHS, TESTS SW1 TO SW4

The values of the effective stress strength parameters are shown at the various compaction conditions in Figure 44 and listed in Table 7. The effective stress angles of friction measured in this study range from 18.9 to 21.4 degrees. The 95 percent confidence intervals for these friction angles, obtained from the individual regression analyses, typically ranged in span from 3 to 7 degrees. In comparison, the measured variation of 2.5 degrees ($21.4 - 18.9 = 2.5$) was not highly significant and could be largely attributed to scatter in the data. Therefore, for the range of compaction and consolidation conditions investigated, the effective stress angle of friction appeared to be invariate with a magnitude of approximately 20 degrees. This variance in friction angle was the result of a complex interaction between compressibility, shear strength and pore water pressure response. Most of the final void ratio versus logarithm of consolidation pressure plots, shown in Figure 30, have very similar slopes. This fact, in combination with the relatively parallel slopes shown on the deviator stress versus final void ratio plots (Figure 35) and on the pore water pressure versus final void ratio plots (Figure 36) resulted in a constant rate of increase in deviator stress and pore water pressure at failure as consolidation pressure increased. For samples compacted dry of optimum, which generally had steeper logarithm of consolidation pressure versus final void ratio curves, the rate of

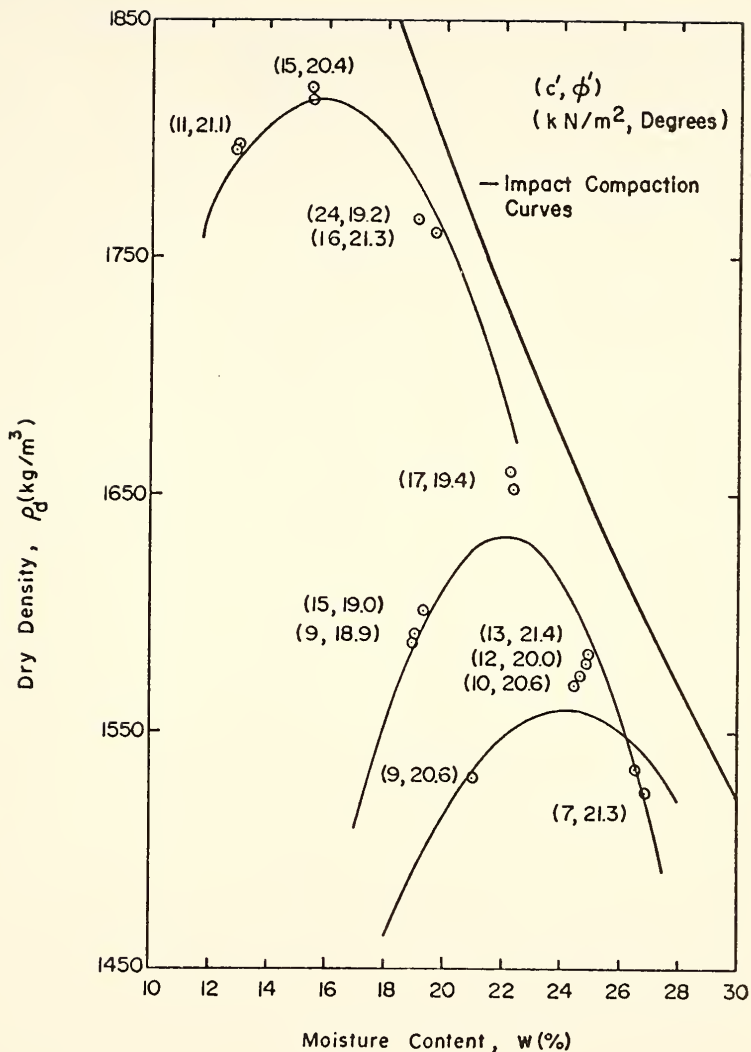


FIGURE 44 EFFECTIVE STRESS STRENGTH PARAMETERS AND ASSOCIATED COMPACTION CONDITIONS

TABLE 7. MEASURED VALUES OF THE EFFECTIVE STRESS STRENGTH PARAMETERS

Test Series	Effective Stress Strength Intercept, c' (kN/m ²)	Effective Stress Friction Angle, ϕ' (Degrees)
MD1(69) to MD4(276)	11	21.1
MO1(69) to MO4(69)	15	20.4
MW1(85) to MW3(276)	24	19.2
MW4(76) to MW6(276)	16	21.3
SD1(69) to SD3(276)	15	19.0
SD4(138) to SD7(138)	9	18.9
SO1(69) to SO4(239)	17	19.4
SW1(69) to SW4(138)	13	21.4
LD1(69) to LD4(69)	9	20.6
LO1(69) to LO3(276)	12	20.0
LO4(69) to LO6(276)	10	20.6
LW1(69) to LW3(276)	7	21.3

increase of deviator stress at failure was greater and that for pore pressure at failure was smaller. These two factors balanced out and resulted in the same friction angle as obtained for the other compaction conditions.

The effective stress strength intercept, c' , ranges from 7 to 24 kN/m^2 . Figure 45 shows that for a constant value of consolidation pressure, c' increased as final void ratio decreased. This means that if two compaction conditions were investigated, the one which achieved lower saturated void ratios for each level of consolidation pressure would have the higher c' value. Figure 45 also shows the large amount of scatter in the data. Much of this scatter was introduced during the extrapolation to the axis intercept from the q_f and p'_f data points prior to determination of c' with equation (3-2). In addition to magnifying the scatter present in the available data, Neter and Wasserman (1974) report that extrapolation is statistically suspect. The statistical problems were avoided by the general assumption of a straight line failure envelope. However, the magnification of the scatter in the data could not be avoided and is demonstrated by the spans of the 95 percent confidence intervals for the individual values of c' . Typical spans for these confidence intervals ranged from 6 to 23 kN/m^2 . As the span of the measured values was only 17 kN/m^2 ($24 - 7 = 17$) considerable scatter in the data was highly probable.

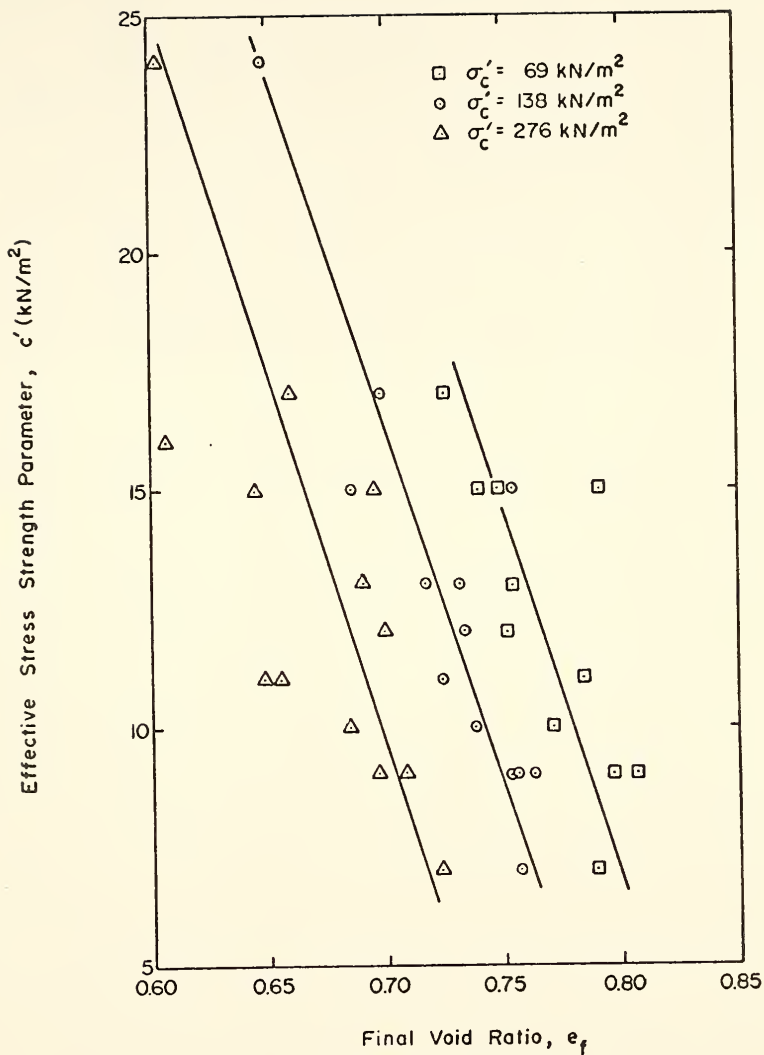


FIGURE 45

EFFECTIVE STRESS STRENGTH PARAMETER
 c' , VERSUS FINAL VOID RATIO

3-6 Statistical Analysis

3-6.1 Linear Regression Analysis

Statistical relationships between dependent and independent variables are often constructed through the use of linear regression analysis. Researchers and practitioners utilize this statistical technique to describe, control and/or predict the behavior of the process of interest. Equation (3-3) represents the general linear regression model,

$$Y_i = \beta_0 + \beta_1 X_{i,1} + \beta_2 X_{i,2} + \dots + \beta_{p-1} X_{i,p-1} + \epsilon_i \quad (3-3)$$

where Y_i is the value of the dependent variable which corresponds to the i^{th} values of the independent variables, $X_{i,1}$ to $X_{i,p-1}$. The error term, ϵ_i , defines this as a statistical rather than a functional model. The model assumes that the ϵ_i values are independent and normally distributed. Good estimates of the regression parameters, β_0 through β_{p-1} , are obtained through application of the method of least squares to the data. This method is based on the premise that the best estimators (b_0 , b_1 , ..., b_{p-1}) are obtained by minimizing the sum of the squares of the error terms ($\sum_{i=1}^n \epsilon_i^2$). Equation (3-4) represents the estimated regression function which is a result of least squares analysis,

$$\hat{Y}_i = b_0 + b_1 X_{i,1} + \dots + b_{p-1} X_{i,p-1} \quad (3-4)$$

where $\hat{Y}_i$ is the estimated value of the dependent variable which corresponds to Y_i , the measured value. Y_i minus $\hat{Y}_i$ is termed a residual. It is actually the sum of squares of the residuals ($\sum (Y_i - \hat{Y}_i)^2 = \sum (e_i^2)$) which is minimized in the method of least squares. Linear regression analysis is a more flexible tool than its name implies, as variables which correlate nonlinearly can be mathematically transformed to show more linear behavior. Products ($X_1 X_2$), inverses ($1/X$), polynomials (X^n) and logarithms ($\log X$) are the transformations most commonly employed. Neter and Wasserman (1974) discuss in depth the basic assumptions, uses and limitations of linear regression analysis.

The Statistical Package for the Social Sciences (SPSS, Nie et al., 1975), part of the software library at Purdue University, provided the programs used in this study to identify meaningful variables and utilize them in linear regression analyses. These programs were combined into a rational procedure, discussed in the following paragraphs, for isolating the best statistical relationships.

The initial stage of the investigation utilized the SPSS routine SCATTERGRAM to plot the dependent variable versus the independent variables and provide quantitative indication of the degree of correlation. Nonlinear plots which appeared susceptible to successful mathematical

transformation were resubmitted to SCATTERGRAM in their transformed state. Linear plots with high coefficients of determination (R^2) were collected to be given early consideration in later regression analyses.

The second stage of the investigation utilized the SPSS routine REGRESSION (STEPWISE option) to form multiple linear regression equations from selected groups of the independent variables. The independent variables chosen initially were those which indicated high linearity during the SCATTERGRAM investigation. If these "good" variables failed to produce a highly satisfactory equation, independent variables with lower coefficients of determination were introduced. Combinations of "poor" variables or "poor" variables with "good" variables result in satisfactory equations if the variables complement each other sufficiently. The STEPWISE option of the REGRESSION routine was chosen as it has the ability to remove as well as add independent variables during construction of an equation. After addition of a new variable to the equation, the program tests all included variables for continued significance. Those no longer contributing significantly are dropped.

The criteria for selection of a satisfactory statistical equation are as follows:

(1) Tests for the overall equation:

- (a) The overall F-test must be satisfactory for the $\alpha = 0.05$ significance level.

- (b) The coefficient of multiple determination (R^2) must be greater than 0.60.
 - (c) The adjusted coefficient of multiple determination (R_a^2) must increase with the addition of each new independent variable.
- (2) Tests for the individual independent variables:
- (a) The F-test for each independent variable must be satisfactory for the $\alpha = 0.05$ significance level while the other independent variable is still in the equation.
 - (b) The 95 percent confidence interval of the estimated regression parameters ($b_1, b_2, \dots, b_{p-1}$) for each independent variable must not include zero.
 - (c) The coefficient of correlation (R) between each pair of independent variables must be less than 0.3.
 - (d) The coefficients of partial determination ($r_{i.jk}^2$) must be significant.
- (3) Tests for the residuals:
- (a) Plots of the standardized residuals ($e_i/\sqrt{MSE}$) versus the independent variables and estimated value of the dependent variable must display constancy of variance and should show good linear fit. (MSE = mean squared error).

- (b) The residuals must be normally distributed random variables.

The residuals were tested for normality through use of the NORPD routine available on Purdue University software. If all criteria were met satisfactorily by more than one equation, the equation with the fewest independent variables was chosen, as long as its value of R_a^2 was within 0.05 of that for the more complicated equation.

3-6.2 Dependent and Independent Variables

Percent volume change due to saturation and consolidation ($\Delta V/V_o, \%$) Skempton's A parameter at failure (A_f) and the effective stress strength parameters (c' and ϕ') were treated as dependent variables in this study. They were correlated with independent variables (discussed in the following paragraphs) which varied in number and in form from one dependent variable to the next.

Table 8 lists the basic independent variables utilized in the analysis of percent volume change due to saturation and consolidation ($\Delta V/V_o, \%$). These basic variables were transformed through the use of logarithms, square roots, squares and inverses. Including combinations (products and divisions) of the basic and transformed variables, over 300 independent variables were analyzed for correlation with $\Delta V/V_o (\%)$.

TABLE 8. BASIC COMPACTION VARIABLES FOR PERCENT VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION

Sample No.	Percent Volume Change Due To Saturation And Consolidation		Compaction Moisture Content $w(\%)$	As-Compacted Dry Density $\rho_d (kg/m^3)$	Kneading Compactor Foot Pressure $p_c (kN/m^2)$		Initial Void Ratio e_o	Initial Degree Of Saturation $S_i(\%)$
	$\Delta V/V_o (\%)$							
MD1 (69)	-14.5		12.94	1797.9	3175		0.558	65.1
MD2 (138)	-10.5		12.89	1795.0	3175		0.560	64.4
ND3 (276)	- 6.1		12.89	1795.0	3175		0.560	64.4
MD4 (276)	- 5.8		12.94	1797.9	3175		0.558	65.1
MO1 (69)	-13.1		15.49	1820.6	2835		0.538	80.6
MO2 (138)	- 9.4		15.54	1816.8	2835		0.541	80.3
NO3 (276)	- 6.7		15.54	1816.8	2835		0.541	80.3
MO4 (69)	-13.7		15.49	1820.6	2835		0.538	80.6
EW1 (85)	- 6.0		19.74	1760.2	2500		0.590	93.5
MW2 (138)	- 4.0		19.08	1766.3	2500		0.585	91.3
MW3 (276)	- 1.2		19.03	1767.3	2500		0.584	91.4
MW4 (76)	- 6.2		19.08	1766.3	2500		0.585	91.3
MW5 (207)	- 2.0		19.08	1767.3	2500		0.584	91.4
MW6 (276)	- 1.1		19.74	1760.2	2500		0.590	93.5
SD1 (69)	- 1.8		19.05	1590.4	818		0.760	70.2
SD2 (138)	- 0.3		19.37	1600.8	818		0.748	72.4
SD3 (276)	3.9		18.98	1588.8	818		0.764	69.8
SD4 (138)	0.5		18.98	1588.8	818		0.764	69.8
SD5 (207)	2.0		19.37	1600.8	818		0.748	72.4
SD6 (276)	2.9		19.05	1590.4	818		0.760	70.2
SD7 (138)	- 0.2		19.05	1590.4	818		0.760	70.2
SO1 (69)	- 2.3		22.22	1658.5	716		0.686	90.7
SO2 (138)	- 0.2		22.39	1651.9	716		0.695	90.2
SO3 (276)	2.1		22.39	1651.9	716		0.695	90.2
SO4 (239)	1.2		22.39	1651.9	716		0.695	90.2

TABLE 8. (continued)

Sample No.	Percent Volume Change Due To Saturation And Consolidation $\Delta V/V_o$ (%)	Compaction Moisture Content w (%)	As-Compacted Dry Density ρ_d (kg/m ³)	Kneading Compactor Foot Pressure p_c (kN/m ²)	Initial Void Ratio e_o	Initial Degree Of Saturation S_i (%)
SW1 (69)	1.1	24.92	1579.0	750	0.773	90.3
SW2 (138)	3.2	24.92	1579.0	750	0.773	90.3
SW3 (276)	4.5	24.93	1581.6	750	0.770	90.6
SW4 (138)	2.2	24.93	1581.6	750	0.770	90.6
LD1 (69)	1.2	20.99	1531.3	409	0.828	70.9
LD2 (138)	4.1	20.99	1531.3	409	0.828	70.9
LD3 (276)	7.2	20.99	1531.3	409	0.828	70.9
LD4 (69)	1.7	20.99	1531.3	409	0.828	70.9
LO1 (69)	1.7	24.50	1569.4	361	0.783	87.5
LO2 (138)	2.5	24.66	1573.3	361	0.779	88.6
LO3 (276)	4.5	24.66	1573.3	361	0.779	88.6
LO4 (69)	0.5	24.66	1573.3	361	0.779	88.6
LO5 (138)	2.5	24.50	1569.4	361	0.783	87.5
LO6 (276)	5.3	24.66	1573.3	361	0.779	88.6
LW1 (69)	2.7	26.85	1523.8	375	0.838	89.7
LW2 (138)	4.4	26.85	1523.8	375	0.838	89.7
LW3 (276)	5.7	26.58	1532.8	375	0.828	89.9

The basic independent variables utilized in the analysis of Skempton's Λ parameter at failure (Λ_f) are listed in Table 9. The estimated compactive prestress ($\hat{p}_s$) values were obtained by inserting the values of compaction moisture content and kneading compactor foot pressure into the predictive equation presented by DiBernardo (1979). Logarithms, square roots, squares, cubes and inverses were utilized to transform the basic independent variables. More than 1000 combinations (products and divisions) of the basic and transformed variables were analyzed for correlation with Λ_f .

Table 10 lists the basic independent variables utilized in the analysis of the effective stress strength parameters (c' and ϕ'). The independent variables were transformed by logarithms, square roots, squares and inverses for analysis of ϕ' , the effective stress friction angle. Products and divisions of the basic and transformed variables increased the number of independent variables to more than 130. For the analysis of c' (the effective stress strength intercept), logarithms, square roots, squares, cubes and inverses were used to transform the basic independent variables. After counting products and divisions of the basic and transformed variables, over 1100 independent variables were investigated for correlation with c' .

TABLE 9. BASIC COMPACTION VARIABLES FOR SKEMPTON'S A PARAMETER AT FAILURE

Sample No.	Skempton's A Parameter At Failure A_f	Compaction Moisture Content $w(\%)$	Kneading			Initial Void Ratio e_o	Initial Degree Of Saturation $S_i(\%)$	Estimated Compactive Prestress $\bar{p}_s$ (kN/m ²)
			As-Compacted Dry Density ρ_d (kg/m ³)	Foot Pressure p_c (kN/m ²)	Compactor Foot Pressure p_c (kN/m ²)			
MD1 (69)	0.37	12.94	1797.9	3175	3175	0.558	65.1	1353
ND2 (138)	0.30	12.89	1795.0	3175	3175	0.560	64.4	1353
ND3 (276)	0.45	12.89	1795.0	3175	3175	0.560	64.4	1353
ND4 (276)	0.36	12.94	1797.9	3175	3175	0.558	65.1	1353
MO1 (69)	0.17	15.49	1820.6	2835	2835	0.538	80.6	895
MO2 (138)	0.08	15.54	1816.8	2835	2835	0.541	80.3	895
MO3 (276)	0.16	15.54	1816.8	2835	2835	0.541	80.3	895
MO4 (69)	0.16	15.49	1820.6	2835	2835	0.538	80.6	895
MO1 (85)	0.11	19.74	1760.2	2500	2500	0.590	93.5	240
MO2 (138)	-0.08	19.08	1766.3	2500	2500	0.585	91.3	240
MO3 (276)	-0.03	19.08	1767.3	2500	2500	0.584	91.4	240
MO4 (76)	0.00	19.08	1766.3	2500	2500	0.585	91.3	240
MO5 (207)	0.06	19.08	1767.3	2500	2500	0.584	91.4	240
MO6 (276)	-0.02	19.74	1760.2	2500	2500	0.590	93.5	240
SD1 (69)	0.24	19.05	1590.4	818	818	0.760	70.2	458
SD2 (138)	0.40	19.37	1600.8	818	818	0.748	72.4	458
SD3 (276)	0.61	18.98	1588.8	818	818	0.764	69.8	458
SD4 (138)	0.60	18.98	1588.8	818	818	0.764	69.8	458
SD5 (207)	0.28	19.37	1600.8	818	818	0.748	72.4	458
SD6 (276)	0.46	19.05	1590.4	818	818	0.760	70.2	458
SD7 (138)	0.52	19.05	1590.4	818	818	0.760	70.2	458
SO1 (69)	-0.15	22.22	1658.5	716	716	0.686	90.7	250
SO2 (138)	-0.01	22.39	1651.9	716	716	0.695	90.2	250
SO3 (276)	0.24	22.39	1651.9	716	716	0.695	90.2	250
SO4 (239)	0.18	22.39	1651.9	716	716	0.695	90.2	250

TABLE 9. (continued)

Sample No.	Skempton's A Parameter At Failure A_f	Compaction Moisture Content $w(\%)$	As-Compacted Dry Density $\rho_d (\text{kg/m}^3)$	Kneading		Initial Void Ratio e_o	Initial Degree Of Saturation $S_i(\%)$	Estimated Compactive Prestress $\hat{p}_s (\text{kN/m}^2)$
				Compactor Foot Pressure $P_c (\text{kN/m}^2)$				
SW1(69)	0.16	24.92	1579.0	750		0.773	90.3	64
SW2(138)	0.30	24.92	1579.0	750		0.773	90.3	64
SW3(276)	0.39	24.93	1581.6	750		0.770	90.6	64
SW4(138)	0.28	24.93	1581.6	750		0.770	90.6	64
LD1(69)	0.39	20.99	1531.3	409		0.828	70.9	286
LD2(138)	0.42	20.99	1531.3	409		0.828	70.9	286
LD3(276)	0.53	20.99	1531.3	409		0.828	70.9	286
LD4(69)	0.35	20.99	1531.3	409		0.828	70.9	286
LO1(69)	0.15	24.50	1569.4	361		0.783	87.5	149
LO2(138)	0.36	24.66	1573.3	361		0.779	88.6	149
LO3(276)	0.52	24.66	1573.3	361		0.779	88.6	149
LO4(69)	0.15	24.66	1573.3	361		0.779	88.6	149
LO5(138)	0.50	24.50	1569.4	361		0.783	87.5	149
LO6(276)	0.50	24.66	1573.3	361		0.779	88.6	149
LW1(69)	0.18	26.85	1523.8	375		0.838	89.7	67
LW2(138)	0.36	26.85	1523.8	375		0.838	89.7	67
LW3(276)	0.47	26.58	1532.8	375		0.828	89.9	67

TABLE 10. BASIC COMPACTION VARIABLES FOR THE EFFECTIVE STRESS STRENGTH PARAMETERS

Test Series	Effective Stress Strength Intercept c' (kN/m ²)	Effective Stress Friction Angle, ϕ' (Degrees)	Compaction Moisture Content w (%)	Dry Density ρ_d (kg/m ³)	Kneading Compactor Foot Pressure p_c (kN/m ²)	Initial Void Ratio e_o	Initial Degree Of Saturation S_i (%)	Estimated Compactive Prestress p'_s (kN/m ²)
MD1 (69) to MD4 (276)	11	21.1	12.92	1796.5	3175	0.559	64.7	1353
MO1 (69) to MO4 (69)	15	20.4	15.52	1818.7	2835	0.540	80.5	895
MM1 (85) to MM3 (276)	24	19.2	19.30	1763.1	2500	0.586	92.1	240
MM4 (76) to MM6 (276)	16	21.3	19.30	1763.1	2500	0.586	92.1	240
SD1 (69) to SD3 (276)	15	19.0	19.12	1592.9	818	0.758	70.7	458
SD4 (138) to SD7 (138)	9	18.9	19.12	1592.9	818	0.758	70.7	458

TABLE 10. (continued)

Test Series	Effective Stress Intercept c' (kN/m ²)	Effective Stress Friction Angle, ϕ' (Degrees)	Compaction Moisture Content w (%)	Dry Density ρ_d (kg/m ³)	Kneading Compactor Foot Pressure p_c (kN/m ²)	Initial Void Ratio e_o	Initial Degree Of Saturation S_i (%)	Estimated Compactive Prestress $\hat{p}_s$ (kN/m ²)
SOL(69) to SO4(239)	17	19.4	22.35	1653.6	716	0.693	90.4	250
SW1(69) to SW4(138)	13	21.4	24.93	1580.3	750	0.772	90.4	64
LD1(69) to LD4(69)	9	20.6	20.99	1531.3	409	0.828	70.9	286
LO1(69) to LO3(276)	12	20.0	24.61	1572.0	361	0.780	88.2	149
LO4(69) to LO6(276)	10	20.6	24.61	1572.0	361	0.780	88.2	149
LW1(69) to LW3(276)	7	21.3	26.76	1526.8	375	0.835	89.7	67

3-6.3 Prediction Model for Volume

Change due to Saturation and Consolidation

Equation (3-5) was the most satisfactory relationship produced during the statistical analysis.

$$\hat{\Delta V}/V_o (\%) = 28.48 - 0.0000136\rho_d^2 + 0.0077S_i\sqrt{\sigma'_c}$$

(3-5)

$\hat{\Delta V}/V_o (\%)$ = estimated value of percent volume change due to saturation and consolidation (%)

ρ_d = as-compacted dry density (kg/m^3)

S_i = initial degree of saturation (%)

σ'_c = isotropic consolidation pressure (kN/m^2)

The statistical data for this equation are shown in Table 11. All statistical criteria discussed in subsection 3-6.1 are surpassed. In particular, the coefficient of determination (R^2) of 0.95 indicates that almost all of the variation in the dependent variable is explained by the variables in the equation. The residuals test satisfactorily as being normally distributed. The standardized residuals, plotted versus $\hat{\Delta V}/V_o (\%)$ and the independent variables in Figures 46, 47 and 48, show some indication of non-linearity of the data. However, the trend is not large enough to jeopardize the integrity of the equation.

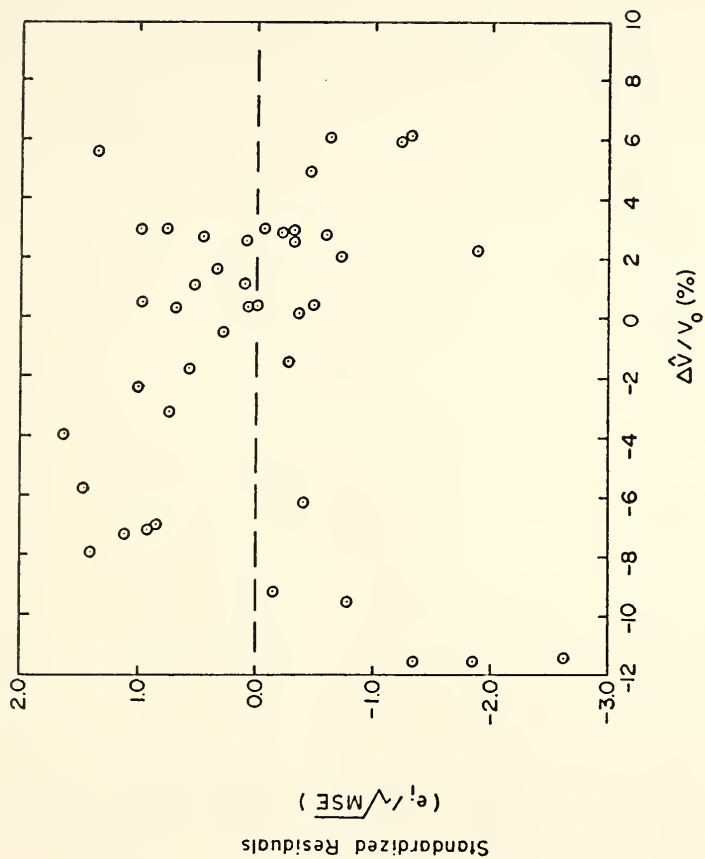
The trends shown by the data and discussed in section 3-3 are well represented by equation (3-5). Figures 49 and 50 show for constant values of saturation and consolidation

TABLE 11. STATISTICAL DATA OF THE PREDICTION EQUATION FOR PERCENT VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION, $\Delta V/V_o(\%)$

Statistical Criteria	Value
Coefficient of Multiple Determination (R^2)	0.95
Adjusted Coefficient of Multiple Determination (R_a^2)	0.95
Overall F-Test	412.49
Partial F-Test	
ρ_d^2	674.39
$S_i \sqrt{\sigma_c^2}$	155.39
Coefficient of Correlation (R)	0.01
Coefficient of Partial Determination*	
$r^2(\Delta \hat{V}/V_o \cdot \rho_d^2 \quad S_i \sqrt{\sigma_c^2})$	0.94
$r^2(\Delta \hat{V}/V_o \cdot S_i \sqrt{\sigma_c^2} \quad \rho_d^2)$	0.89

* $r^2(Y \cdot X_1 \quad X_2)$ = coefficient of partial determination between Y and X_1 when X_2 is in the equation

FIGURE 46 STANDARDIZED RESIDUALS VERSUS ESTIMATED PERCENT VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION



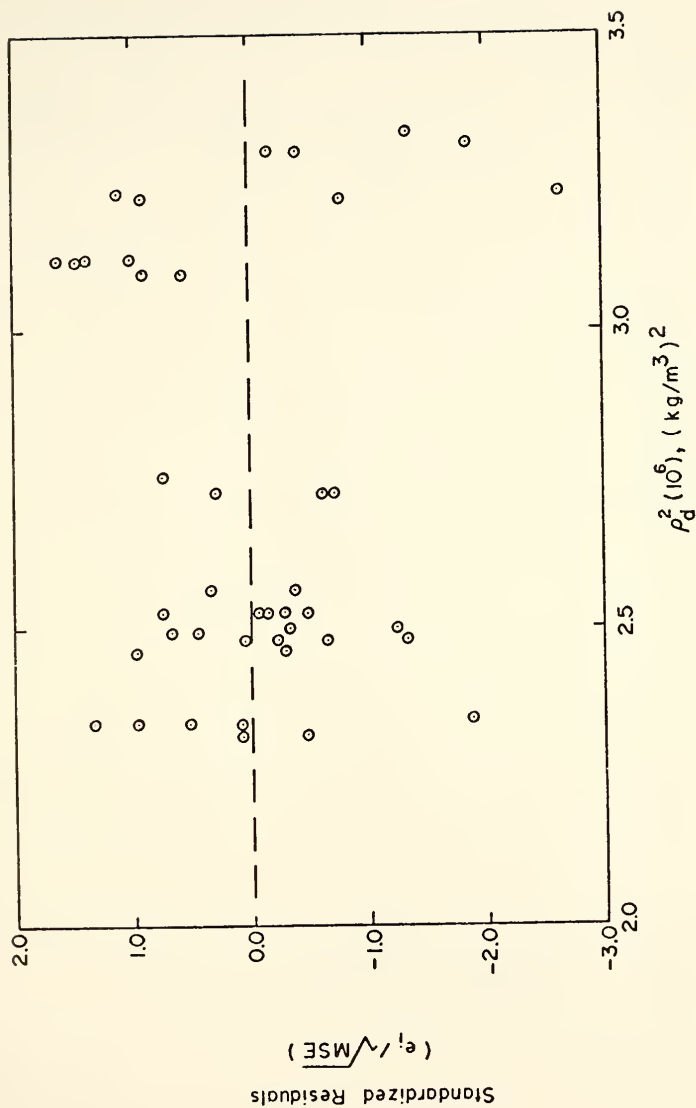


FIGURE 47 STANDARDIZED RESIDUALS VERSUS AN INDEPENDENT VARIABLE OF THE PREDICTION EQUATION FOR VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION

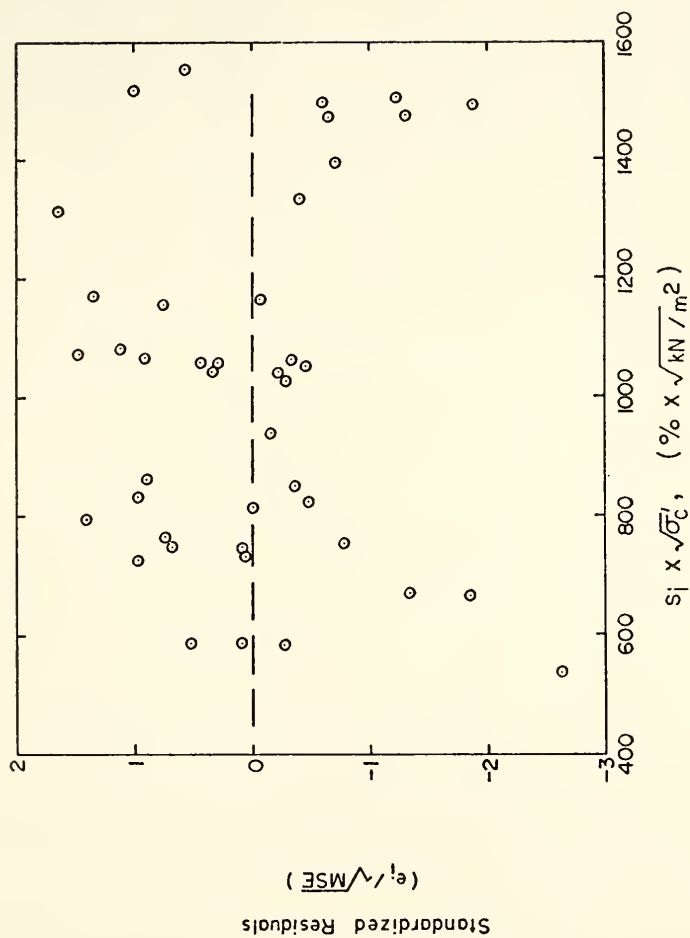


FIGURE 48 STANDARDIZED RESIDUALS VERSUS AN INDEPENDENT VARIABLE OF THE PREDICTION EQUATION FOR VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION

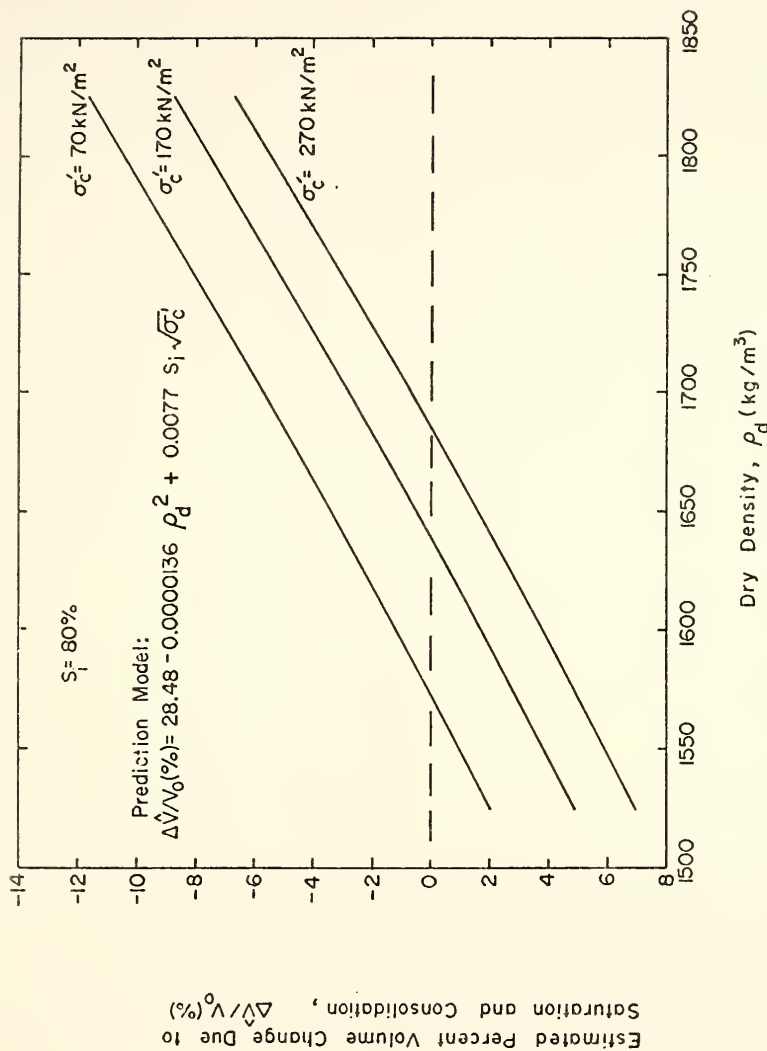


FIGURE 49 PREDICTION OF PERCENT VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION FOR AN INITIAL SATURATION OF 80%

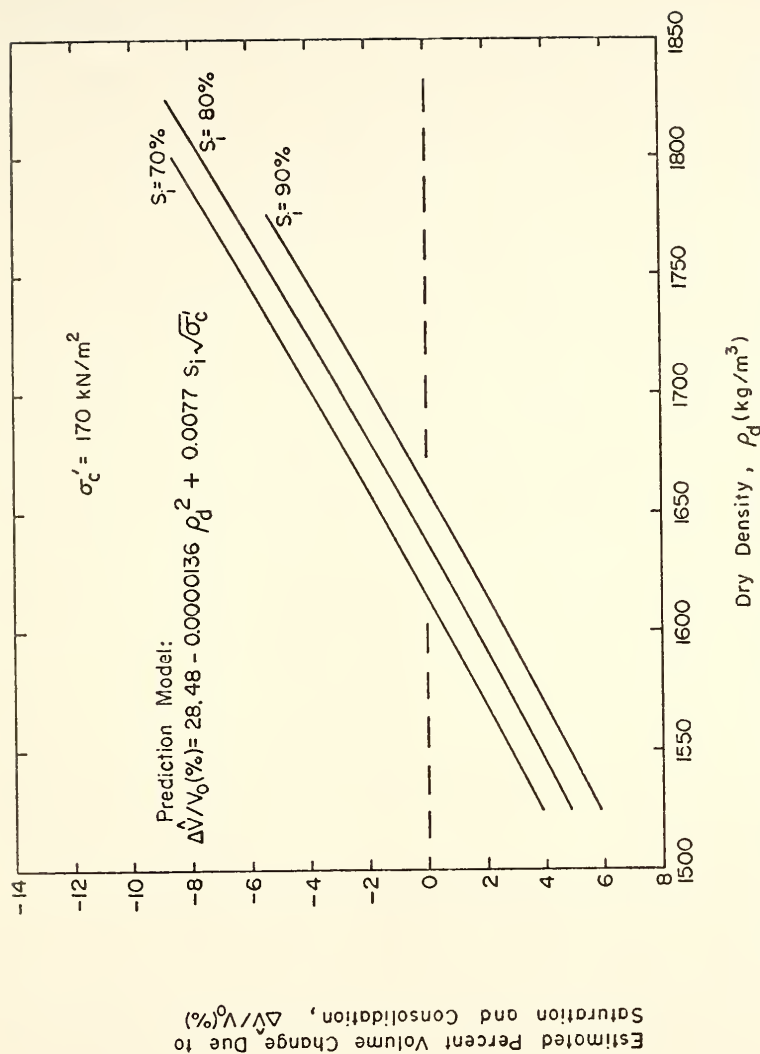
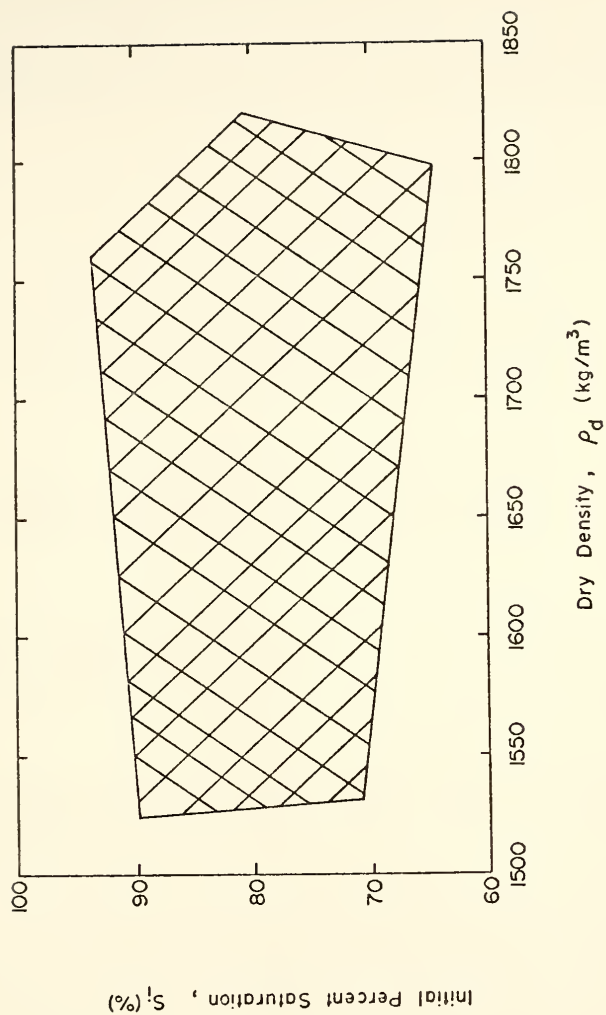


FIGURE 50 PREDICTION OF PERCENT VOLUME CHANGE DUE TO SATURATION² AND CONSOLIDATION AT A CONSOLIDATION PRESSURE OF 170 kN/m²

pressure that $\hat{\Delta V}/V_0(\%)$ decreases (more net swell or less net compression) as dry density increases. This is a result of the greater swell potential of compacted fine grained soils at higher densities. The concept of "water deficiency", presented by Lambe (1958) can be utilized to explain the trend of greater swell with increasing dry density for a given consolidation pressure and degree of saturation. The closer packing of soil particles attained with increasing dry density results in a greater "water deficiency" per particle if the percent saturation remains constant or decreases. Upon inundation, the great uptake of water per particle necessary to satisfy the greater "water deficiency" per particle becomes more severe. When inundated, expansion of the more highly depressed double layer results in more swell. Figure 49 shows that for constant values of dry density and initial degree of saturation, $\hat{\Delta V}/V_0(\%)$ increases (less net swell or more net compression) as consolidation pressure increases.

The prediction nomograph for $\hat{\Delta V}/V_0(\%)$, shown in Figure 52, allows determination without calculation of the appropriate value of $\hat{\Delta V}/V_0(\%)$ for a given set of compaction and consolidation conditions. Before using the nomograph it is necessary to enter Figure 51 with the initial degree of saturation and dry density to ensure that they both lie within the range of conditions investigated. It is also required that the consolidation pressure be between 69 and



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AND DRY DENSITY OBSERVATIONS

FIGURE 51

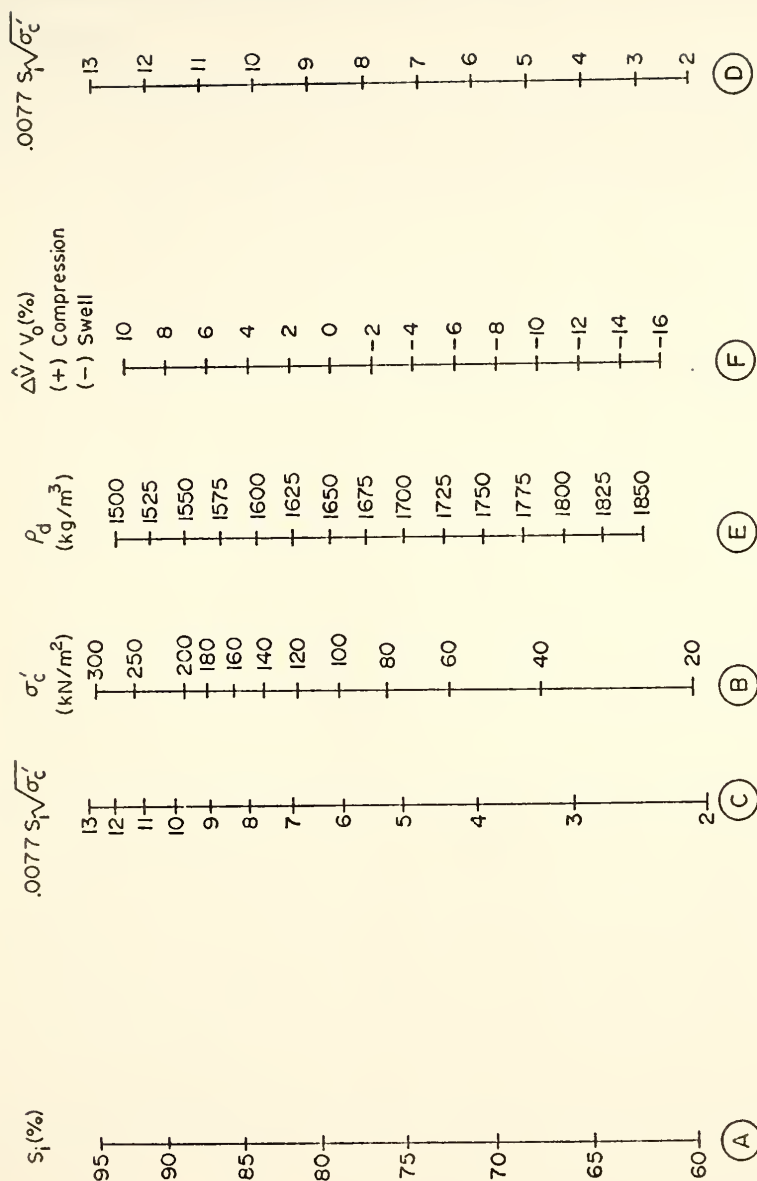


FIGURE 52 PREDICTION NOMOGRAPH FOR PERCENT VOLUME CHANGE DUE TO SATURATION AND CONSOLIDATION, $\Delta \hat{V}/V_0(\%)$

276 kN/m². The correct procedure for using the nomograph follows.

- (1) Enter the initial degree of saturation in column A and the isotropic consolidation pressure in column B, and connect the two points with a straight line.
- (2) Read the value of $0.0077 S_i \sqrt{\sigma'_C}$ on column C and transfer it to column D.
- (3) Enter the value of as-compacted dry density in column E and connect it with a straight line to the value of $0.0077 S_i \sqrt{\sigma'_C}$ in column D.
- (4) Read the value of $\hat{\Delta V}/V_o$ (%) from column F.

For example, a sample compacted to a dry density of 1650 kg/m³ and a moisture content of 22.4 percent has an initial degree of saturation (S_i) of 90 percent. A line drawn between the S_i value of 90 percent on the A column and the value of consolidation pressure (σ'_C) to be used on the B column (assumed equal to 80 kN/m² for this example) produces an intercept of 6.22 on column C. A line drawn between the dry density value of 1650 on column E and the value of $0.0077 S_i \sqrt{\sigma'_C}$ transferred from column C to column D (6.22) intercepts column F at -2.30. The value of $\hat{\Delta V}/V_o$ (%) for this example is therefore -2.30 and represents net swell.

3-6.4 Prediction Model for Skempton's A

Parameter at Failure

Equation (3-6) was the most satisfactory relationship produced during the statistical analysis of A_f .

$$\hat{A}_f = 1.79 - 0.00011 \rho_d \sqrt{S_i} + 1.28 \sigma'_c / \rho_d \quad (3-6)$$

$\hat{A}_f$ = estimated value of Skempton's A parameter at failure

ρ_d = as-compacted dry density (kg/m^3)

S_i = initial degree of saturation (%)

σ'_c = isotropic consolidation pressure (kN/m^2)

The statistical data for this equation, shown in Table 12, are all satisfactory. The multiple coefficient of determination (R^2) value of 0.71 is acceptable, but not highly satisfactory. It is likely that much of the unexplained variation in A_f can be attributed to the scatter caused largely by temperature fluctuations. The scatter and its causes are discussed in detail in section 3-4. The residuals test satisfactorily as being normally distributed. The standardized residuals, plotted versus $\hat{A}_f$ and the independent variables in Figures 53, 54 and 55 indicate constancy of variance and linearity of the data.

Equation (3-6) shows the same trends isolated and discussed in section 3-4. Figure 56 shows that for constant values of dry density and initial degree of saturation an increase in the consolidation pressure produces an increase

TABLE 12. STATISTICAL DATA OF THE PREDICTION EQUATION FOR SKEMPTON'S A PARAMETER AT FAILURE, A_f

Statistical Criteria	Value
Coefficient of Multiple Determination (R^2)	0.71
Adjusted Coefficient of Multiple Determination (R_a^2)	0.69
Overall F-Test	46.97
Partial F-Test	
$\rho_d \sqrt{S_i}$	74.01
σ'_c / ρ_d	15.59
Coefficient of Correlation (R)	-0.06
Coefficient of Partial Determination*	
$r^2(\hat{A}_f \cdot \rho_d \sqrt{S_i} \quad \sigma'_c / \rho_d)$	0.66
$r^2(\hat{A}_f \cdot \sigma'_c / \rho_d \quad \rho_d \sqrt{S_i})$	0.28

* $r^2(Y \cdot X_1 \ X_2)$ = coefficient of partial determination between Y and X_1 when X_2 is in the equation.

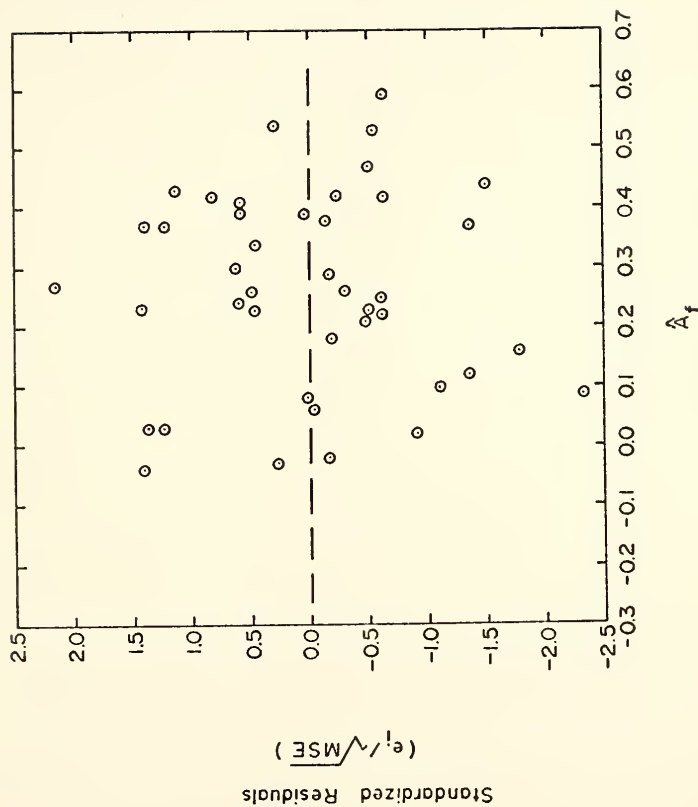


FIGURE 53

STANDARDIZED RESIDUALS VERSUS ESTIMATED VALUE
OF SKEMPTON'S A PARAMETER AT FAILURE

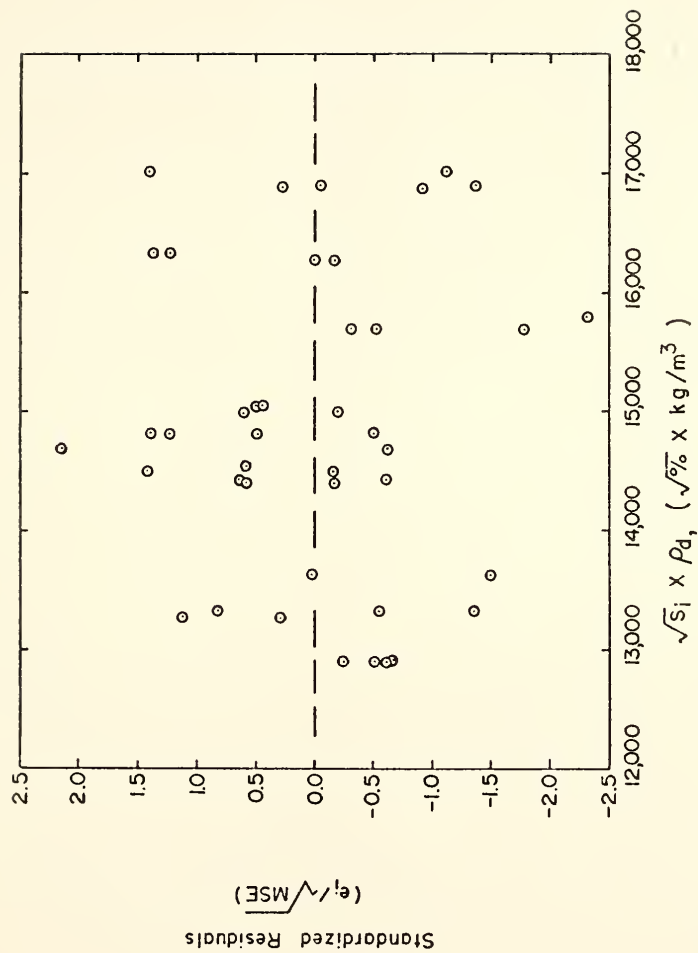


FIGURE 54 STANDARDIZED RESIDUALS VERSUS AN INDEPENDENT VARIABLE OF THE PREDICTION EQUATION FOR SKEMPTON'S A_f PARAMETER AT FAILURE

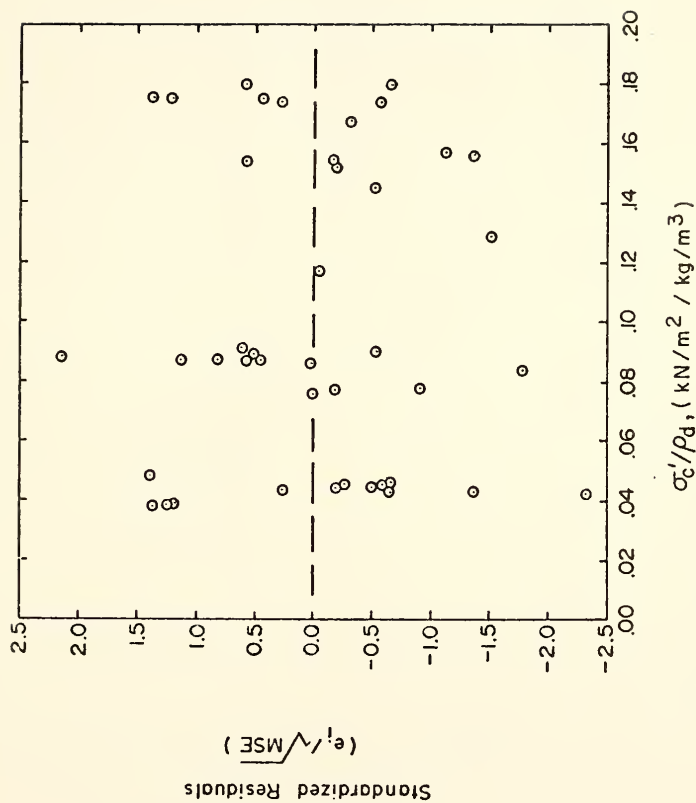


FIGURE 55 STANDARDIZED RESIDUALS VERSUS AN INDEPENDENT VARIABLE OF THE PREDICTION EQUATION FOR SKEMPTON'S A PARAMETER AT FAILURE

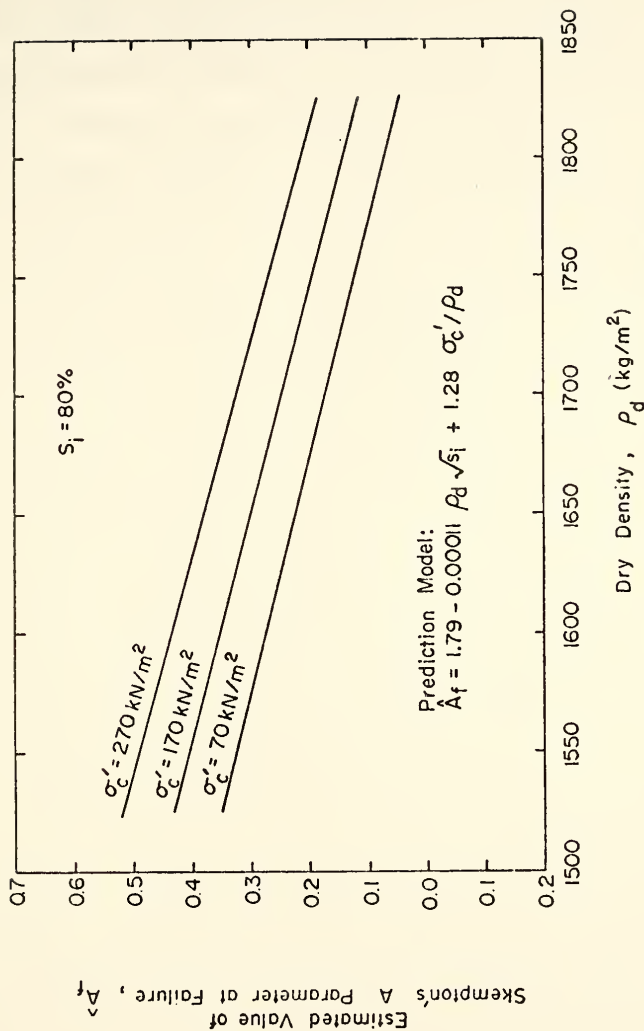


FIGURE 56

PREDICTION OF SKEMPTON'S A PARAMETER AT FAILURE
 FOR AN INITIAL SATURATION OF 80%

in A_f . This is in agreement with the results presented by Henkel (1956) for remolded clay, as an increase in consolidation pressure represents a decrease in overconsolidation ratio. Figure 8 illustrated the increase in A_f which resulted from a decrease in overconsolidation ratio. Figures 56 and 58 show that for constant values of consolidation pressure and initial degree of saturation, $\hat{A}_f$ decreases as dry density increases. This behavior is analogous to that displayed by sand samples sheared at various relative densities. Loose samples compress during shear (or develop positive pore pressures if sheared undrained) whereas very dense samples dilate (or develop negative pore pressures if sheared undrained.) Figures 26, 27 and 28 illustrate the validity of this analogy. For constant values of consolidation pressure and initial degree of saturation the saturation and consolidation process does not alter the order of the samples in terms of their dry densities. For example, a sample compacted to a low energy Proctor dry density is sheared at a lower dry density than a sample compacted at a standard Proctor dry density. The low energy Proctor sample then produces a higher A_f value when sheared as its lower density (higher void ratio) makes it more amenable to compression than the denser standard Proctor sample. Figure 57 illustrates theoretically the effect of differences in post saturated-consolidated dry density. For a constant value of consolidation pressure (lines A or B for example), an

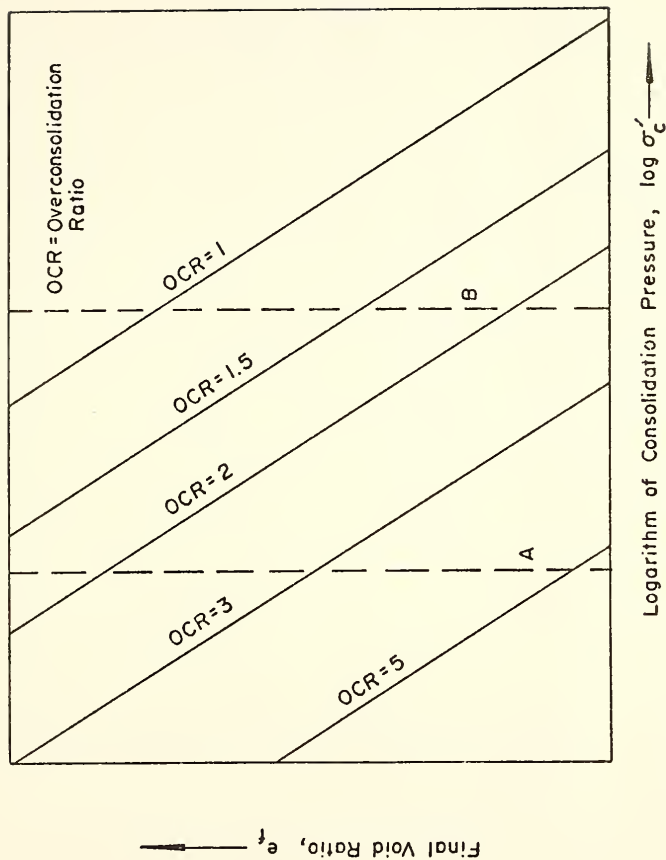


FIGURE 57

EFFECT OF OVERCONSOLIDATION RATIO ON THE
RELATIONSHIP BETWEEN FINAL VOID RATIO
AND CONSOLIDATION PRESSURE

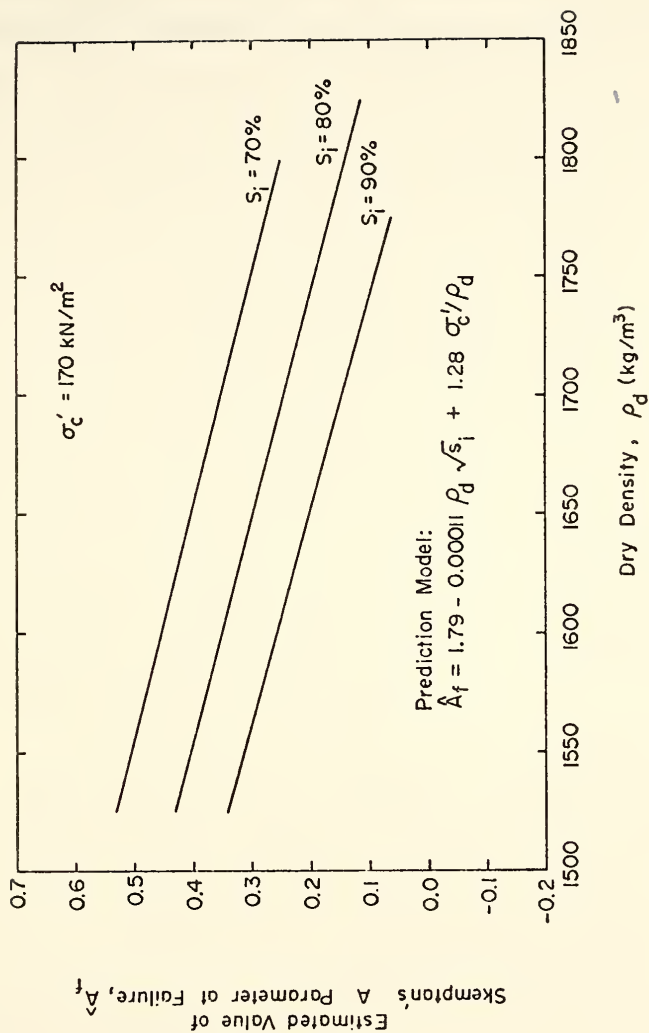


FIGURE 58

PREDICTION OF SKEMPTON'S A PARAMETER AT FAILURE
 FOR A CONSOLIDATION PRESSURE OF 170 kN/m²

increase in final void ratio (a decrease in dry density) results in a decreased value of overconsolidation ratio (OCR). As noted earlier and shown in Figure 8, a decrease in OCR produces an increase in A_f . Figure 58 shows for constant values of consolidation pressure and as-compacted dry density that an increase in initial degree of saturation produces a decrease in $\hat{A}_f$. This trend can be attributed to the decreasing swell potential obtained with higher percent saturation. For two samples compacted to equal dry densities and unequal degrees of saturation, the one with the low degree of saturation will swell the most (or compress the least) for a given consolidation pressure and be more amenable to compression during shear. It will therefore produce a higher value of A_f than the sample compacted to a higher degree of saturation.

The prediction nomograph for $\hat{A}_f$ is shown in Figure 59. Before entering the nomograph it is necessary to plot the values of initial degree of saturation and dry density on Figure 51 to ensure that they lie within the range of conditions investigated. Also, the consolidation pressure must be between 69 and 276 kN/m². The correct procedure for using the nomograph follows:

- (1) Enter the initial degree of saturation in column A and the as-compacted dry density in column B, and connect the two points with a straight line.
- (2) Read the value of $0.00011\rho_d\sqrt{S_1}$ on column C.

FIGURE 59 PREDICTION NOMOGRAPH FOR SKEMPTON'S A PARAMETER AT FAILURE, $\hat{A}_f$

- (3) Enter the as-compacted dry density in column B and the consolidation pressure in column D, and connect the two points with a straight line.
- (4) Read the value of $1.28 \sigma'_C / \rho_d$ on column E and transfer it directly to column F.
- (5) Connect the value of $0.00011 \rho_d \sqrt{S_i}$ on column C with the value of $1.28 \sigma'_C / \rho_d$ on column F with a straight line.
- (6) Read the value of $\hat{A}_f$ on column G.

For example, a sample compacted to a dry density of 1650 kg/m^3 and a moisture content of 22.4 percent has an initial degree of saturation (S_i) of 90 percent. A line drawn between the S_i value of 90 percent on column A and the dry density value of 1650 kg/m^3 on column B produces a value of $(0.00011 \rho_d \sqrt{S_i}) = -1.72$ on column C. A line drawn between the dry density value of 1650 kg/m^3 on column B and the consolidation pressure value of 80 kN/m^2 (chosen arbitrarily for the example) on column D produces an intercept value of $(1.28 \sigma'_C / \rho_d = 0.062)$ on column E. A line drawn between 0.062 on column F and -1.72 on column C produces an $\hat{A}_f$ value of 0.11 on column G.

3-6.5 Prediction Model for the

Effective Stress Friction Angle, ϕ'

Equation (3-7) was the most satisfactory relationship produced during the analysis of ϕ' .

$$\phi' = 20.1^{\circ} \pm 1.3^{\circ} \quad (3-7)$$

The basic and transformed independent variables discussed in subsection 3-6.2 provided no satisfactory relationships. This result was not unexpected, however, due to greater possible variation in friction angle due to scatter in the data than was actually measured between the values for all of the compaction conditions. The scatter in the data was discussed in more detail in section 3-5.

3-6.6 Prediction Model for the

Effective Stress Strength Intercept, c'

Equation (3-8) was the most satisfactory relationship produced during the statistical analysis of c' .

$$\hat{c}' = 1.71 - 3.83 w \log e_o \quad (3-8)$$

$\hat{c}'$ = estimated value of the effective stress strength intercept (kN/m^2)

w = compaction moisture content (%)

e_o = initial void ratio

The statistical data for this equation are all satisfactory and are shown in Table 13. The multiple coefficient of determination (R^2) value of 0.63 is acceptable, but not highly satisfactory. The lack of a better correlation can be attributed to the initial scatter in the q versus p' data and its magnification during extrapolation to the intercept. The scatter and its causes are discussed in more detail in section 3-5. The standardized residuals, plotted

TABLE 13. STATISTICAL DATA OF THE PREDICTION EQUATION FOR THE EFFECTIVE STRESS STRENGTH INTERCEPT, c'

Statistical Criteria	Value
Coefficient of Multiple Determination (R^2)	0.63
Adjusted Coefficient of Multiple Determination (R_a^2)	0.59
Overall F-Test	16.92

versus the independent variable in Figure 60, indicate constancy of variance and linearity of the data. The residuals also test satisfactorily as being normally distributed.

Figure 61 shows that for a constant value of compaction moisture content, an increase in initial void ratio produces a decrease in $\hat{c}'$. In section 3-5 it was noted that an increase in the final void ratio for a constant value of consolidation pressure also produced a decrease in c' . These trends are in agreement because the order of the samples in terms of void ratio is not altered by the process of saturation and consolidation for a given consolidation pressure. Reference to Figure 57 shows that an increase in final void ratio at a given consolidation pressure implies a decrease in overconsolidation ratio. As shown theoretically in Figure 62, this results in a decrease in c' . Figure

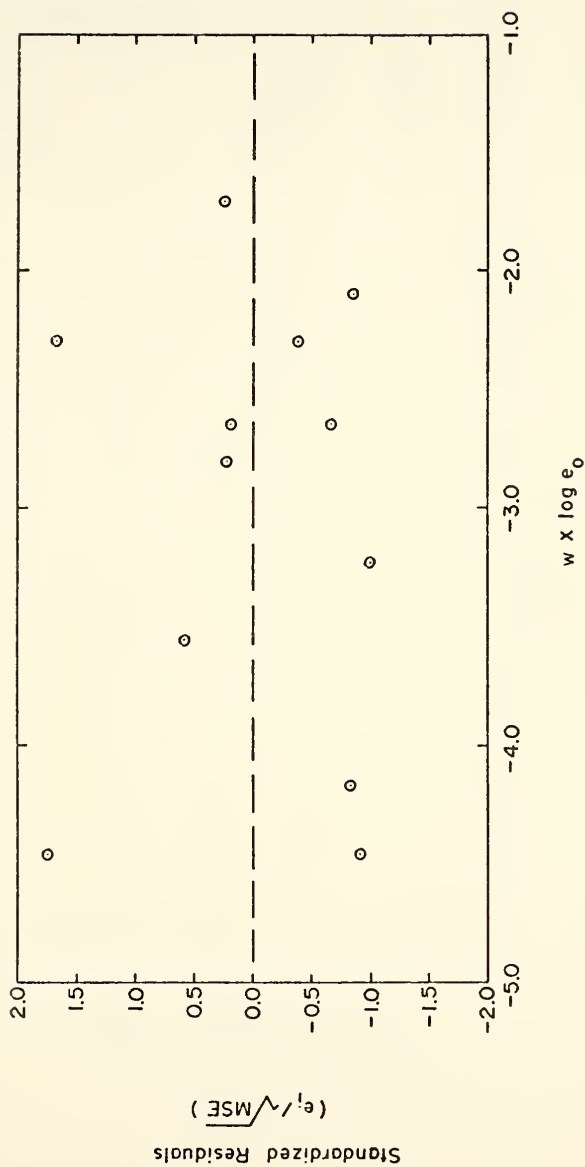


FIGURE 60 STANDARDIZED RESIDUALS VERSUS THE INDEPENDENT VARIABLE OF THE PREDICTION EQUATION FOR THE EFFECTIVE STRESS STRENGTH INTERCEPT

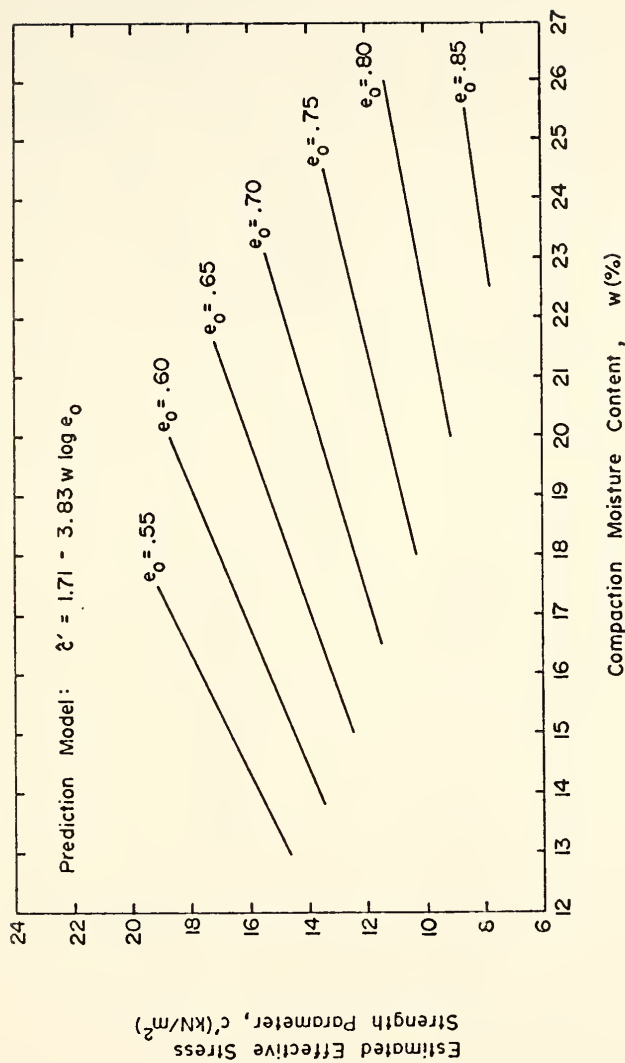


FIGURE 6I

PREDICTION OF EFFECTIVE STRESS STRENGTH PARAMETER, c'

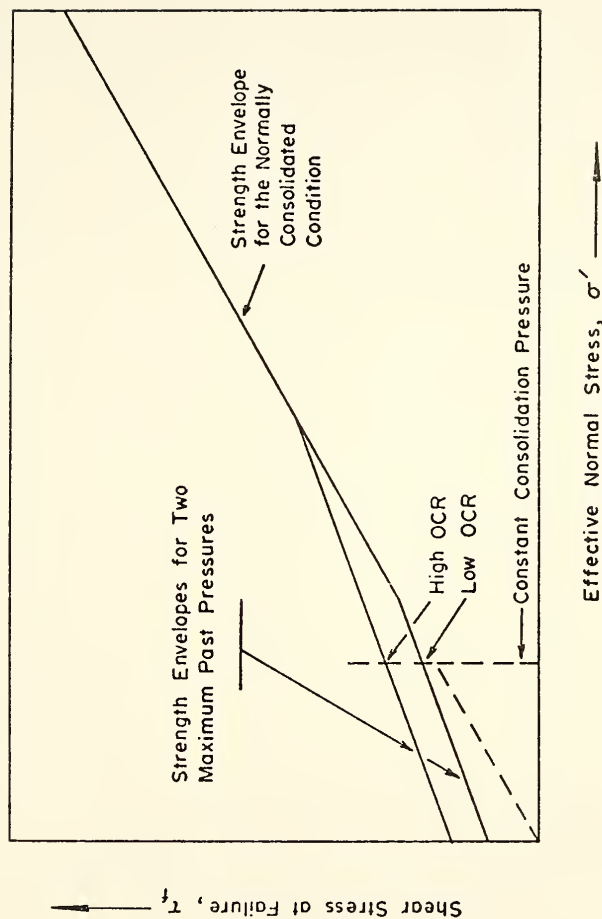


FIGURE 62

EFFECT OF MAXIMUM PAST PRESSURE ON THE EFFECTIVE STRESS STRENGTH INTERCEPT

61 shows for a constant value of initial void ratio that an increase in the compaction moisture content produces an increase in c' . This behavior can be attributed to the greater swell potential of the samples compacted to the lower compaction moisture contents. As the compaction moisture content increases the final void ratio attained for a given consolidation pressure decreases. A lower void ratio implies a higher overconsolidation ratio or a higher maximum past pressure and a higher value of c' .

Before using Figure 61 for prediction purposes it is necessary to enter the values of initial void ratio and compaction moisture content in Figure 63 to ensure that they lie within the range of conditions investigated. As the range of c' values is small and the scatter for the data is significant, the designer may simply elect to place a rough lower bound on c' , such as 10 kN/m^2 , and utilize it for all conditions.

3-7 Comparison of Strength Behavior of Samples Consolidated to Anisotropic and Isotropic States of Stress

Brooker and Ireland (1965) reported that the value of K_0 (K_0 = lateral effective stress/vertical effective stress) increases as overconsolidation ratio increases. For a compacted soil mass with a consistent amount of compaction per lift and adequate lateral confinement, the K_0 dependency on OCR indicates a decrease in K_0 with increasing

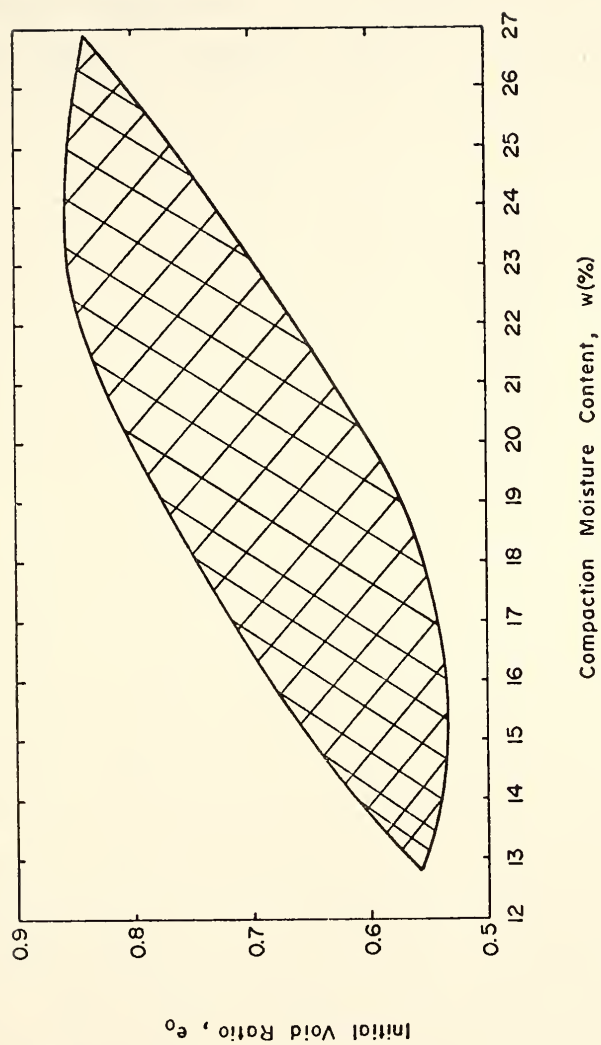


FIGURE 63 JOINT REGION OF INITIAL VOID RATIO AND MOISTURE CONTENT OBSERVATIONS

depth. To simulate true K_0 conditions in a triaxial cell requires rather sophisticated test equipment to monitor and maintain constant the radial dimension of the sample during consolidation or rebound. However, the lack of control over radial expansion following sampling and preceding shear casts doubt on the efficacy of K_0 determination in a triaxial cell. Also, it is highly improbable that K_0 conditions are actually maintained in a compacted embankment due to its geometry. The actual K values probably lie between that for active earth pressure (K_a) and no lateral strain (K_0). For these reasons sophisticated tests were not utilized and a single K value of 0.67 was arbitrarily chosen for testing.

Four samples, compacted at optimum moisture content to a low energy Proctor dry density, were consolidated at the chosen ratio of radial to axial stress. Each was then sheared undrained to provide insight into possible behavioral differences between samples consolidated to anisotropic and isotropic stress states. Table B2 lists the compaction, saturation and consolidation data for the four samples. Figure 64 shows the stress versus strain and pore pressure versus strain plots. The rounded, smooth curves are similar to those obtained for the isotropically consolidated samples. Table 14 lists the data at failure for each sample. The stress paths and failure envelope are shown in Figure 65. The effective stress strength parameters were determined through linear regression analysis in the same manner

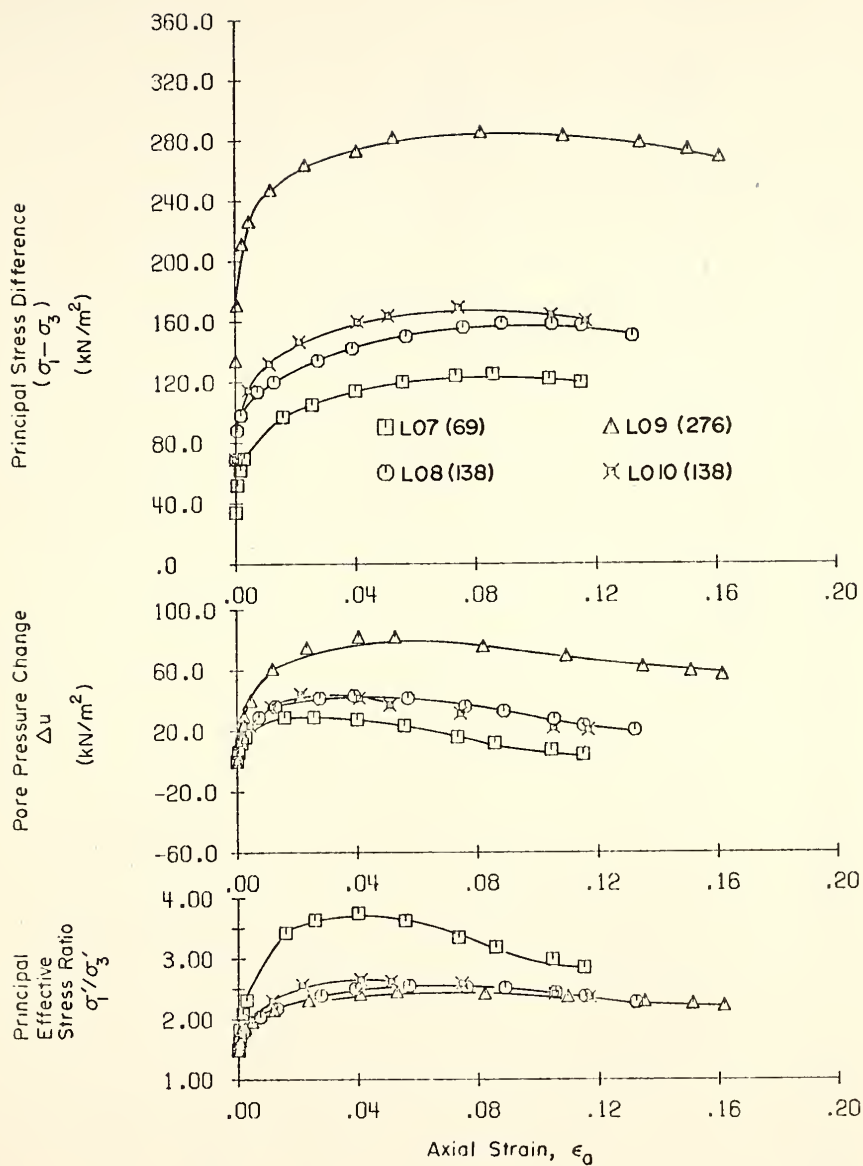


FIGURE 64 CAU RESULTS, TESTS L07 TO L010

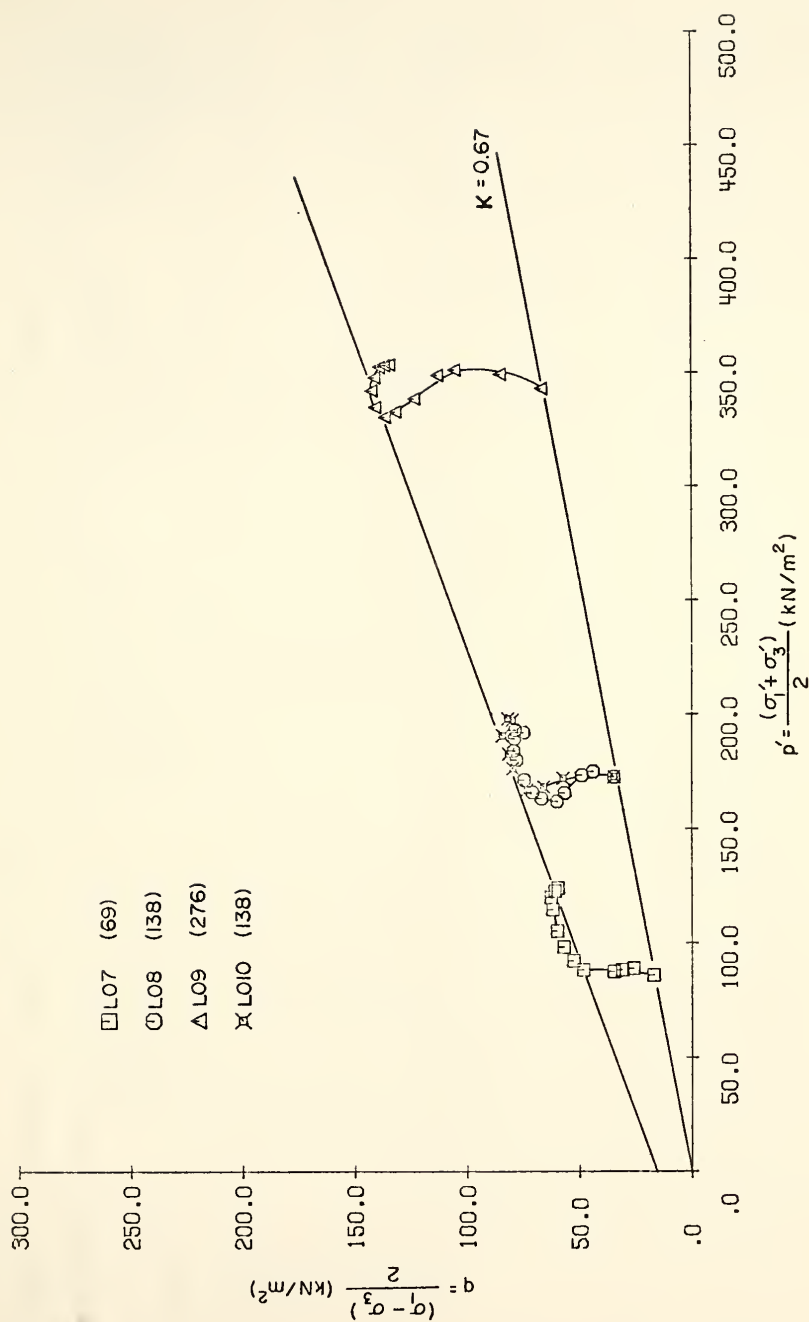


FIGURE 65 EFFECTIVE STRESS PATHS, TESTS L07 TO L010

TABLE 14. FAILURE CONDITIONS FOR ANISOTROPICALLY CONSOLIDATED SAMPLES

Sample No.	Principal Stress Difference After Consolidation, ² ($\sigma_1 - \sigma_3$) _c (kN/m ²)	Principal Stress Difference At Failure ($\sigma_1 - \sigma_3$) _f (kN/m ²)	Pore Pressure Change At Failure, ² Δu_f (kN/m ²)	Skempton's A Parameter At Failure A_f
LO7(69)	34	125	12	0.13
LO8(138)	69	159	33	0.37
LO9(276)	134	286	76	0.50
LO10(138)	69	169	32	0.32

as those for the isotropically consolidated samples. The effective stress strength intercept (c') is 16 kN/m^2 and the friction angle (ϕ') is 21.6 degrees. Both parameters are greater in magnitude than those obtained for isotropic consolidation (c' values of 12 and 10 kN/m^2 and ϕ' values of 20.0 and 20.6 degrees respectively). As the differences are of a magnitude which could easily be attributed to the scatter in the data it is not absolutely certain that anisotropic consolidation produces higher strength parameters, although it appears to do so. If this observation holds for the remainder of the compaction conditions, the use of isotropic stress states for consolidation produces slightly conservative effective stress strength parameters. Holtz and Krizek (1971) presented the results of undrained triaxial tests on clay consolidated from a slurry which indicated that anisotropic consolidation produced more conservative strength parameters. The most important observation, however, is that the measured differences are small in both studies.

4 - CONCLUSIONS AND RECOMMENDATIONS

4-1 Conclusions

- (1) After accounting for measurement errors, the back pressure requirements for saturation of a compacted highly plastic clay (St. Croix) appear to be satisfactorily predicted by the equation presented by Lowe and Johnson (1960).
- (2) The long term volume change behavior of compacted Saint Croix clay is largely a function of the swell potential of its as-compacted condition, with depth of overburden (consolidation pressure) exerting a large but secondary influence. This is borne out by the following trends:
 - (a) An increase in as-compacted dry density (ρ_d) for constant values of initial degree of saturation (S_i) and consolidation pressure (σ'_c) produces a decrease in percent volume change ($\Delta V/V_o$, %) due to saturation and consolidation (more net swell or less net compression).
 - (b) With the exception of the two collapse prone compaction conditions, an increase in the value of S_i for constant values of ρ_d and σ'_c produces an increase in $\Delta V/V_o$ (%).

These trends show agreement with the results of swell tests presented by Holtz and Gibbs (1956) and Parcher and Liu (1965).

- (3) Due to the strong influence of swell potential on the long term volume change behavior of compacted Saint Croix clay, the process of saturation and consolidation greatly reduces the range of dry density and moisture content values from those created during compaction. This moderating influence is very important as it places limitations on shear behavior and strength variation.
- (4) Skempton's A parameter at failure (A_f) is largely a function of the final void ratio (e_f) attained with the selected consolidation pressure and is probably related to the degree of overconsolidation. As e_f is a function of the percent volume change due to saturation and consolidation ($\Delta V/V_o, \%$), A_f is greatly affected by the swell potential of the as-compacted condition. The following trends are observed:
 - (a) A decrease in the initial void ratio (e_o) for constant values of initial degree of saturation (S_i) and consolidation pressure (σ'_c) produces a decrease in final void ratio (e_f), which in turn produces a decrease in A_f .
 - (b) A decrease in S_i for constant values of e_o and σ'_c produces an increase in e_f , which in turn produces an increase in A_f .

- (c) An increase in σ'_c for constant values of e_o and S_i probably represents a reduction in overconsolidation ratio and produces an increase in A_f .
- (5) Estimated compactive prestress ($\hat{p}_s$) as defined by DiBernardo (1979) and kneading compactor foot pressure (p_c) are inadequate representations of the maximum past pressure remembered by the soil after saturation. DiBernardo's parameters are greater in magnitude for a given dry density dry of optimum than wet of optimum. However, the trend noted in conclusion 4b for A_f implies a greater maximum past pressure for compaction wet of optimum. It appears that the swelling process may partially destroy the "memory" of a compacted soil. The lack of variation in the stress versus strain curve shapes for the various compaction conditions may also be attributed to this effect.
- (6) The effective stress friction angle, ϕ' , for the range of compaction conditions investigated for the Saint Croix clay is essentially a constant value with an average of 20.1 degrees and a range of ± 1.3 degrees.
- (7) The effective stress strength intercept, c' , is largely a function of the final void ratio (e_f) attained with a selected consolidation pressure and is probably related to overconsolidation ratio and the maximum past pressure remembered by the soil. As e_f is a function of the percent volume change due to

saturation and consolidation ($\Delta V/V_o, \%$) or c' is greatly affected by the swell potential of the as-compacted condition.

- (8) The average value of the effective stress strength intercept, c' , is approximately 15 kN/m^2 for compacted Saint Croix clay and its range is approximately $\pm 8 \text{ kN/m}^2$. As the range of variation in c' is small and errors induced during extrapolation to the intercept of the failure envelope can be significantly large, the use of a single lower bound value would not induce significant error in stability analyses.
- (9) The following equations, obtained through the use of linear regression analysis, were determined to be the best predictive equations for volume change due to saturation and consolidation ($\Delta V/V_o, \%$), Skempton's A parameter at failure (A_f) and the effective stress strength intercept (c').

$$(a) \quad \Delta V/\hat{V}_o(\%) = 28.48 - 0.0000136 \rho_d^2 + 0.0077 S_i \sqrt{\sigma_c'}$$

$$(\text{See Figures 49 and 50}) \quad R^2 = 0.95$$

$$(b) \quad \hat{A}_f = 1.79 - 0.00011 \rho_d \sqrt{S_i} + 1.28 \sigma_c' / \rho_d$$

$$(\text{see Figures 56 and 58}) \quad R^2 = 0.71$$

$$(c) \quad \hat{c}' = 1.71 - 3.83 w (\log e_o)$$

$$(\text{See Figure 61}) \quad R^2 = 0.63$$

In the above equations, ρ_d = as-compacted dry density, S_i = initial degree of saturation, σ_c' = isotropic consolidation pressure, w = compaction moisture content

and e_o = initial void ratio. The low coefficient of determination (R^2) of the predictive equation for λ_f can be partially attributed to variation in pore pressure and deviator stress at failure induced by temperature changes during undrained shear. The low coefficient of determination (R^2) of the predictive equation for c' can be attributed to the small range of intercept values measured and the highly significant error induced during extrapolation of the failure envelope to the axis intercept.

- (10) Consolidation to an anisotropic rather than an isotropic state of stress appears to produce small increases in the effective stress strength parameters. However, this statement is based on a very limited data base and requires verification through further testing.

4-2 Recommendations for Future Research

- (1) The effects of the field compaction variables on percent volume change due to saturation and consolidation, Skempton's λ parameter at failure and the effective stress strength parameters for Saint Croix clay should be investigated and correlated with the effects of laboratory compaction.
- (2) A laboratory study of the long term properties and behavior of a clay with low swell potential should be conducted and compared with the results of this study.

- (3) A study of the pore size distributions of compacted clays before and after swell should be conducted and related to shear behavior.
- (4) A measure of the net or effective compactive energy for different compaction modes should be devised and related to the swell potential and long term behavior of the various compaction conditions.
- (5) A study of the effect of the ratio of lateral to vertical effective stresses for compacted soil after saturation and consolidation should be conducted and utilized in a study of long term shear behavior.

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APPENDICES

NOTE: The following Appendices are not included in this
copy of the Report:

	<u>Pages</u>
Appendix A - Impact Compaction Data	192-193
Appendix B - Compaction, Saturation and Consolidation Data	194-197
Appendix C - Undrained Shear Data	198-252
Appendix E - Purdue University Negative Numbers	270-271

Persons desiring a copy of any or all of the above indicated
Appendices may obtain a copy for the cost of duplication upon
request to:

Joint Highway Research Project
Civil Engineering Building
Purdue University
West Lafayette, Indiana 47907

Appendix D
Supplementary Figures

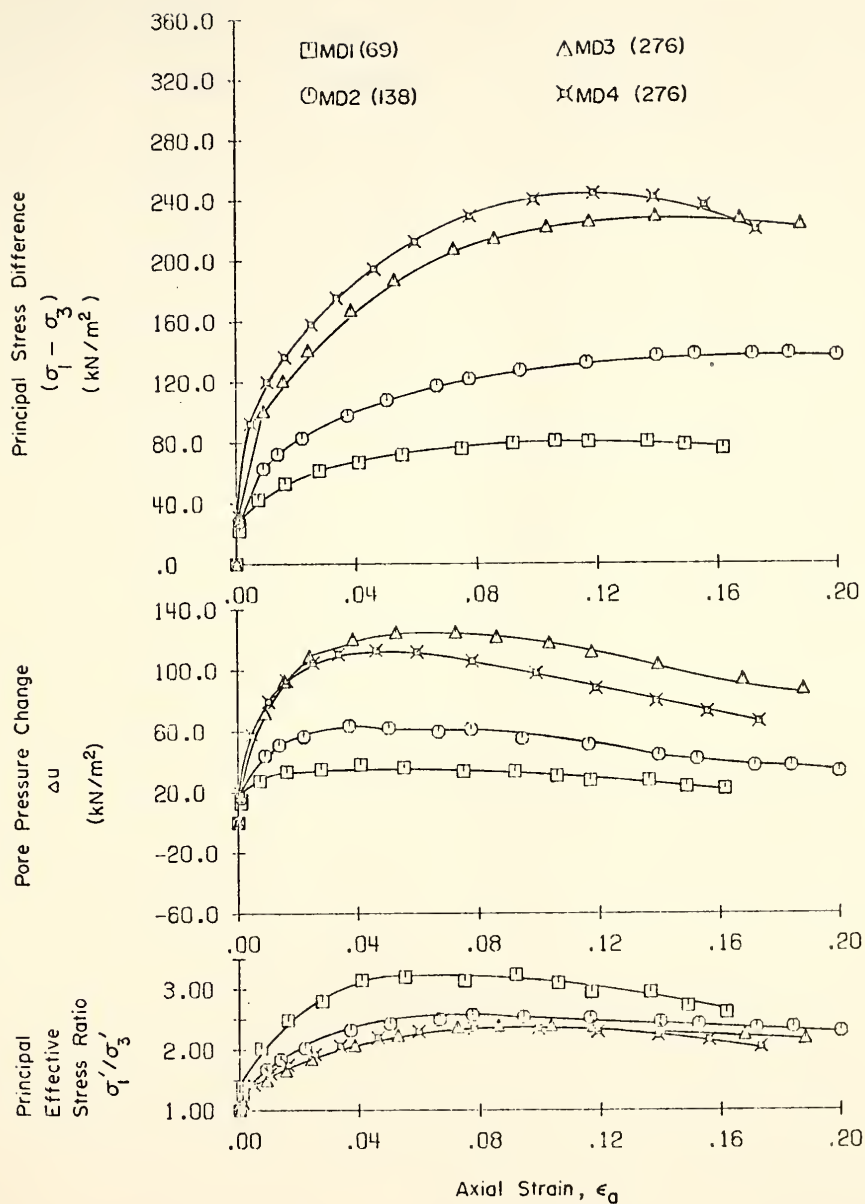


FIGURE D1 CIU RESULTS, TEST MDI TO MD4

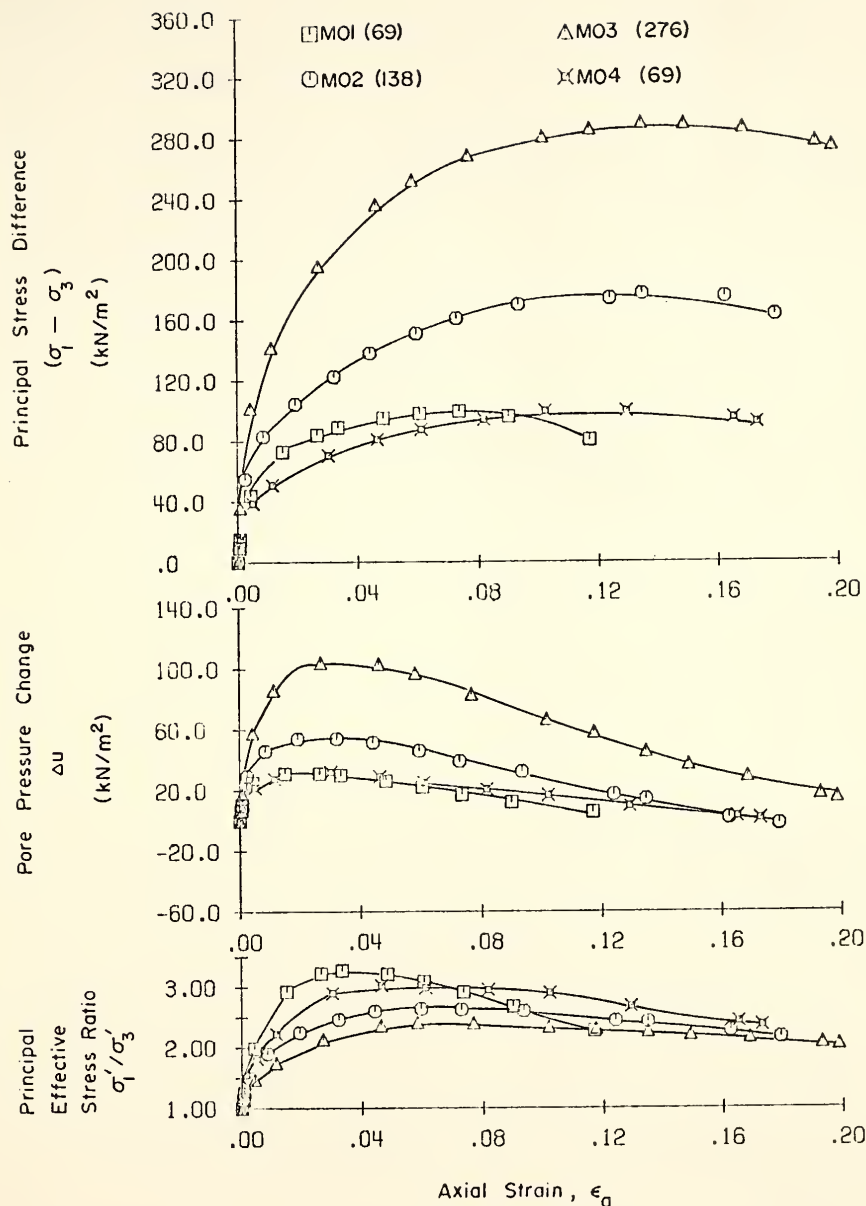


FIGURE D2 CIU RESULTS, TESTS MO1 TO MO4

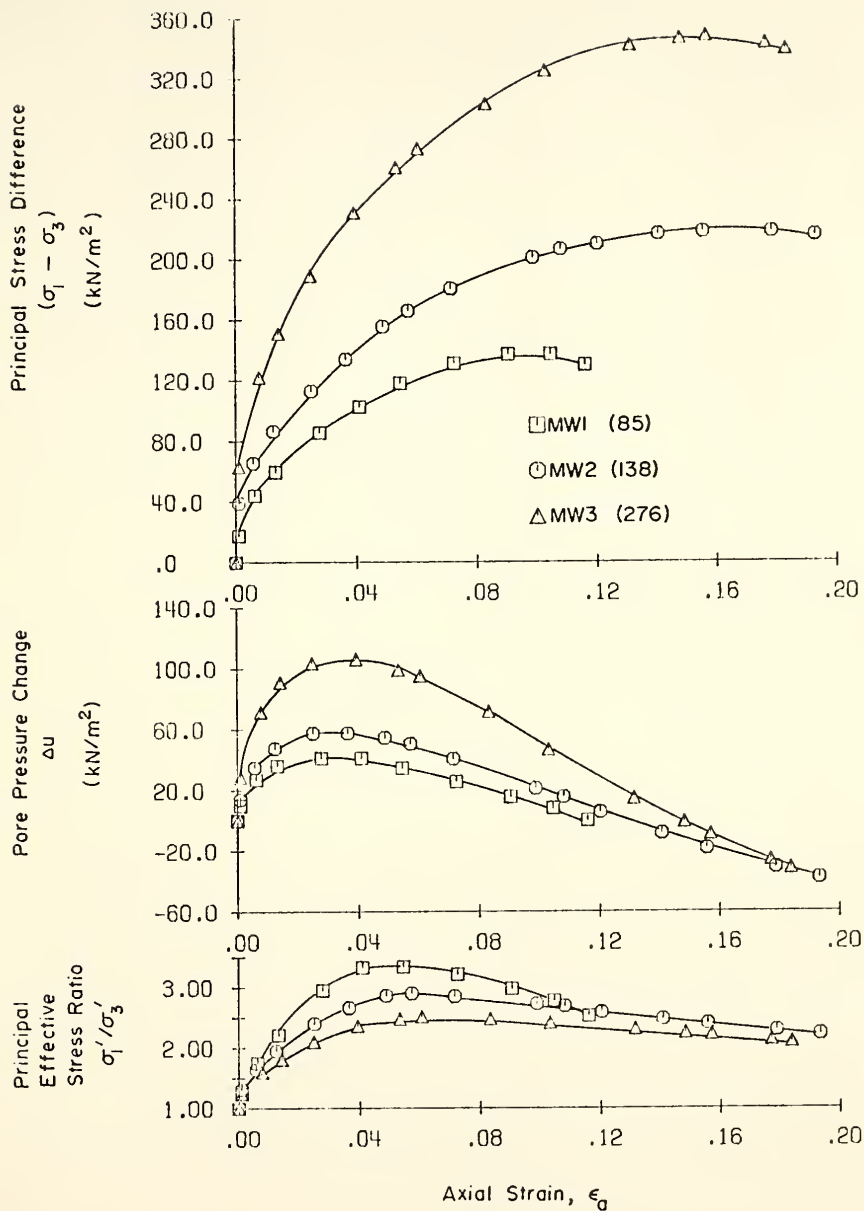


FIGURE D3 CIU RESULTS, TESTS MWI TO MW3

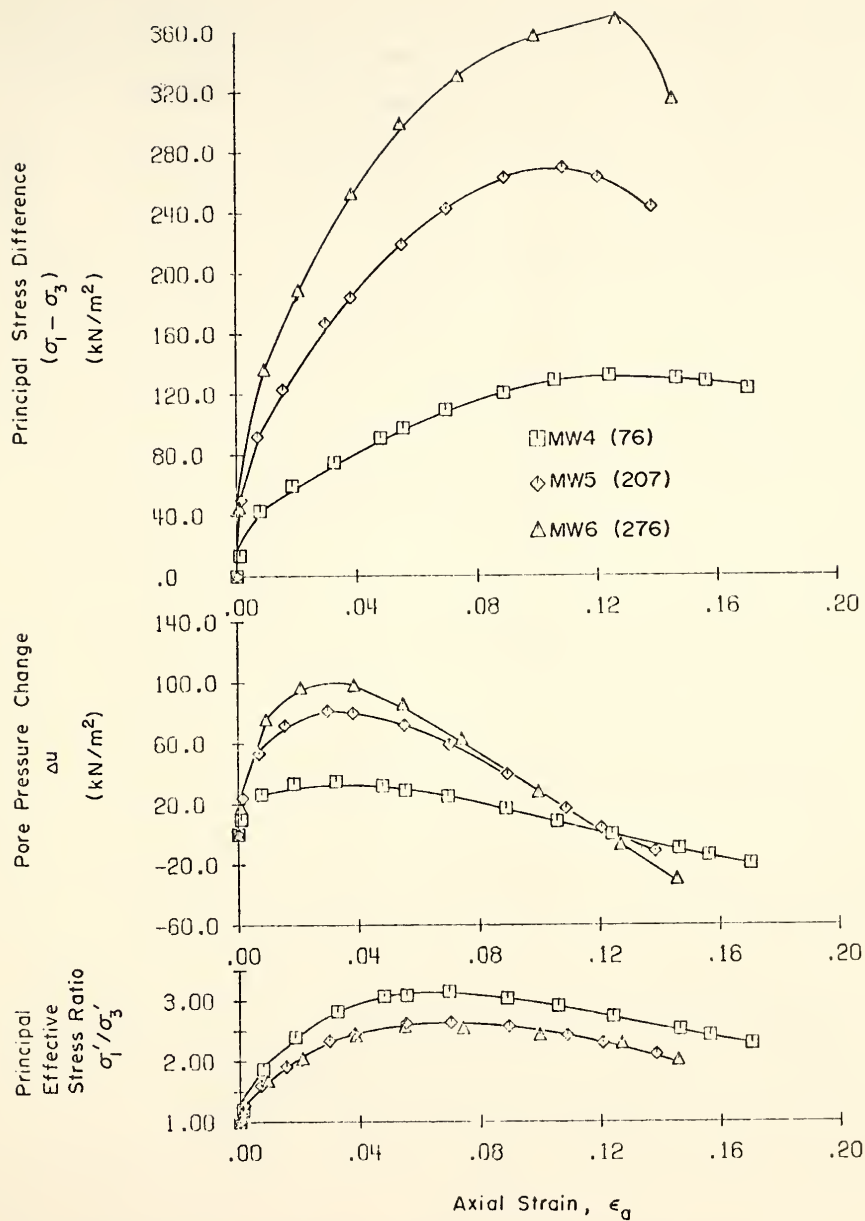


FIGURE D4 CIU RESULTS, TESTS MW4 TO MW6

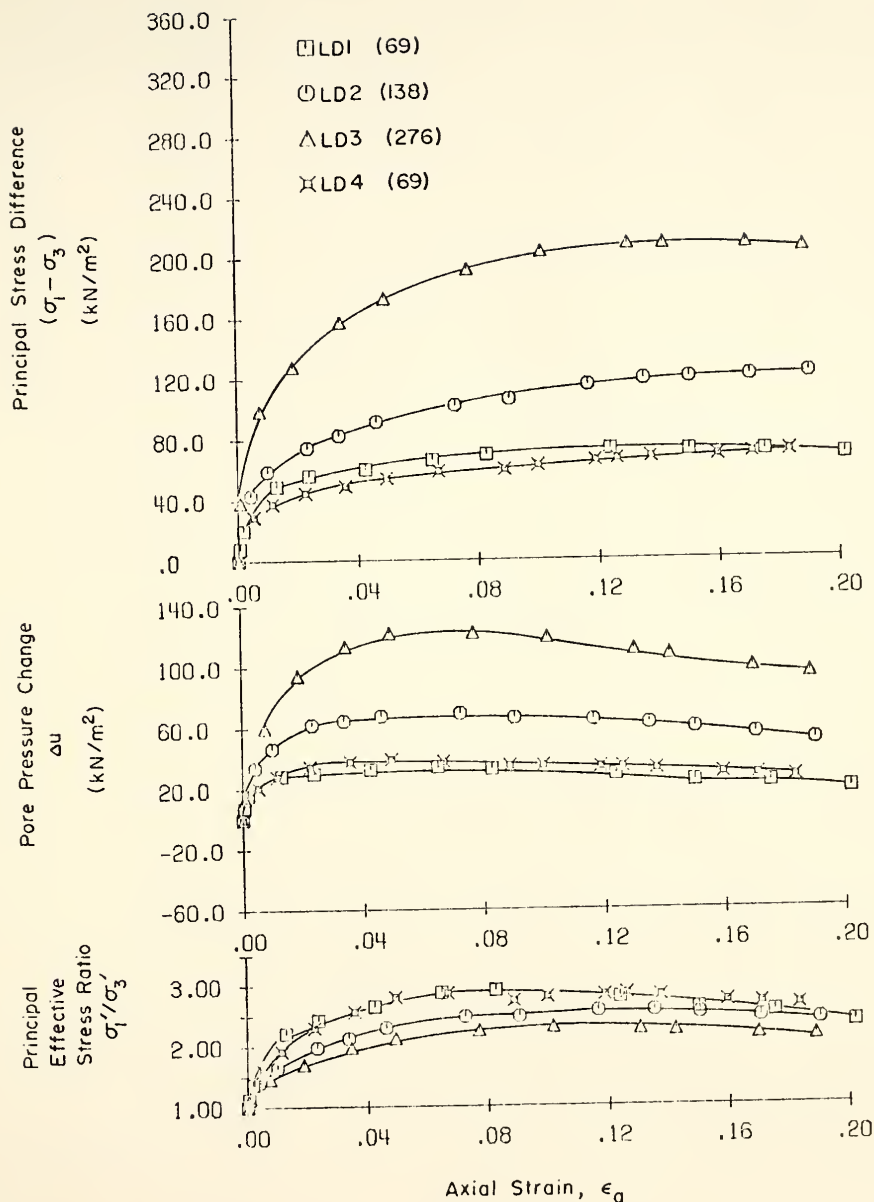


FIGURE D5 CIU RESULTS, TESTS LDI TO LD4

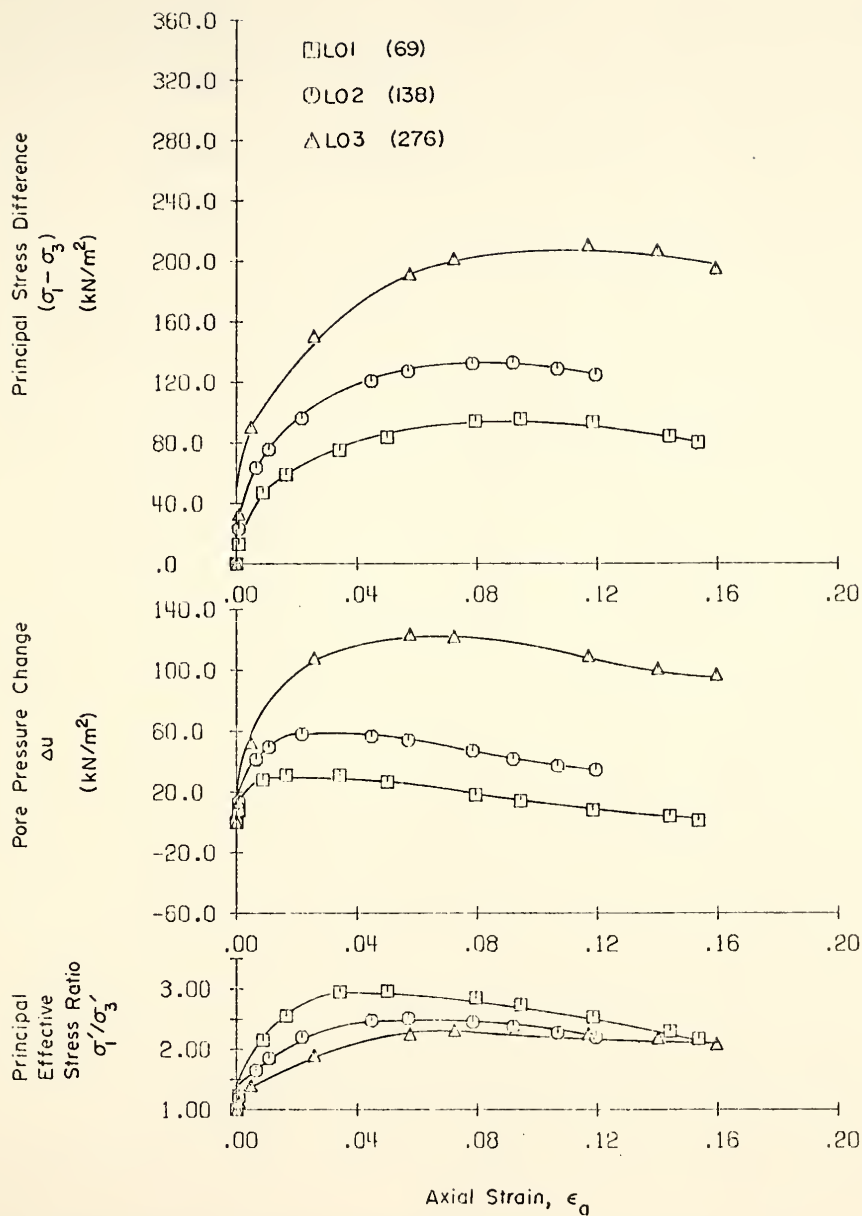


FIGURE D6 CIU RESULTS, TESTS LO1 TO LO3

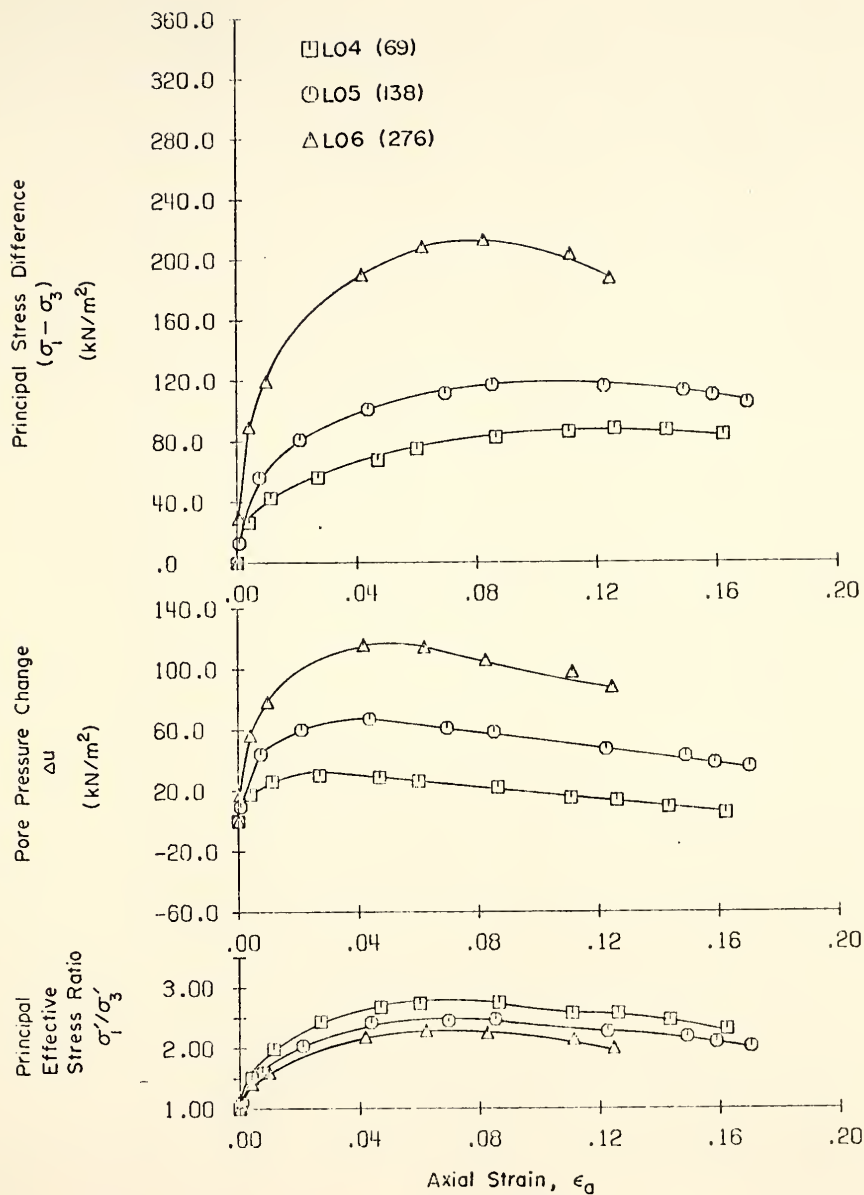


FIGURE D7 CIU RESULTS, TESTS L04 TO L06

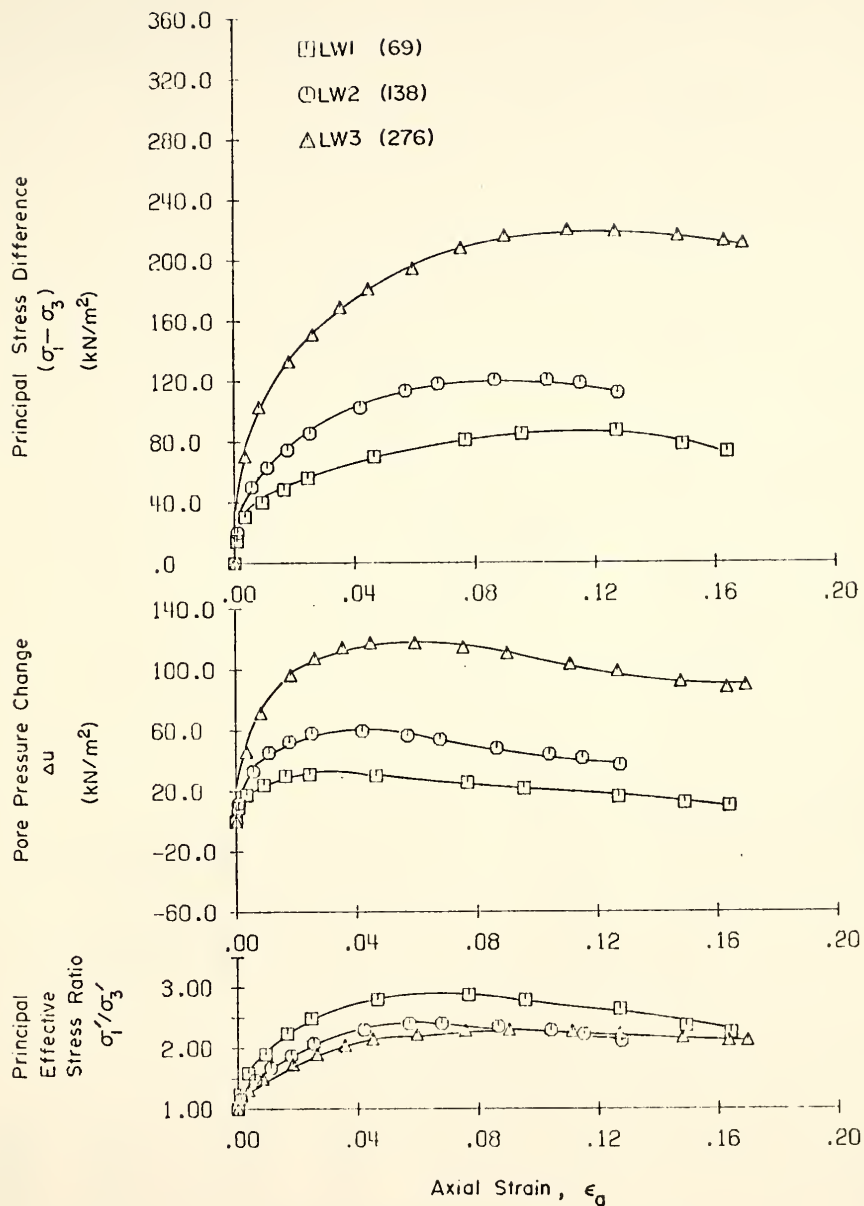


FIGURE D8 CIU RESULTS, TESTS LWI TO LW3

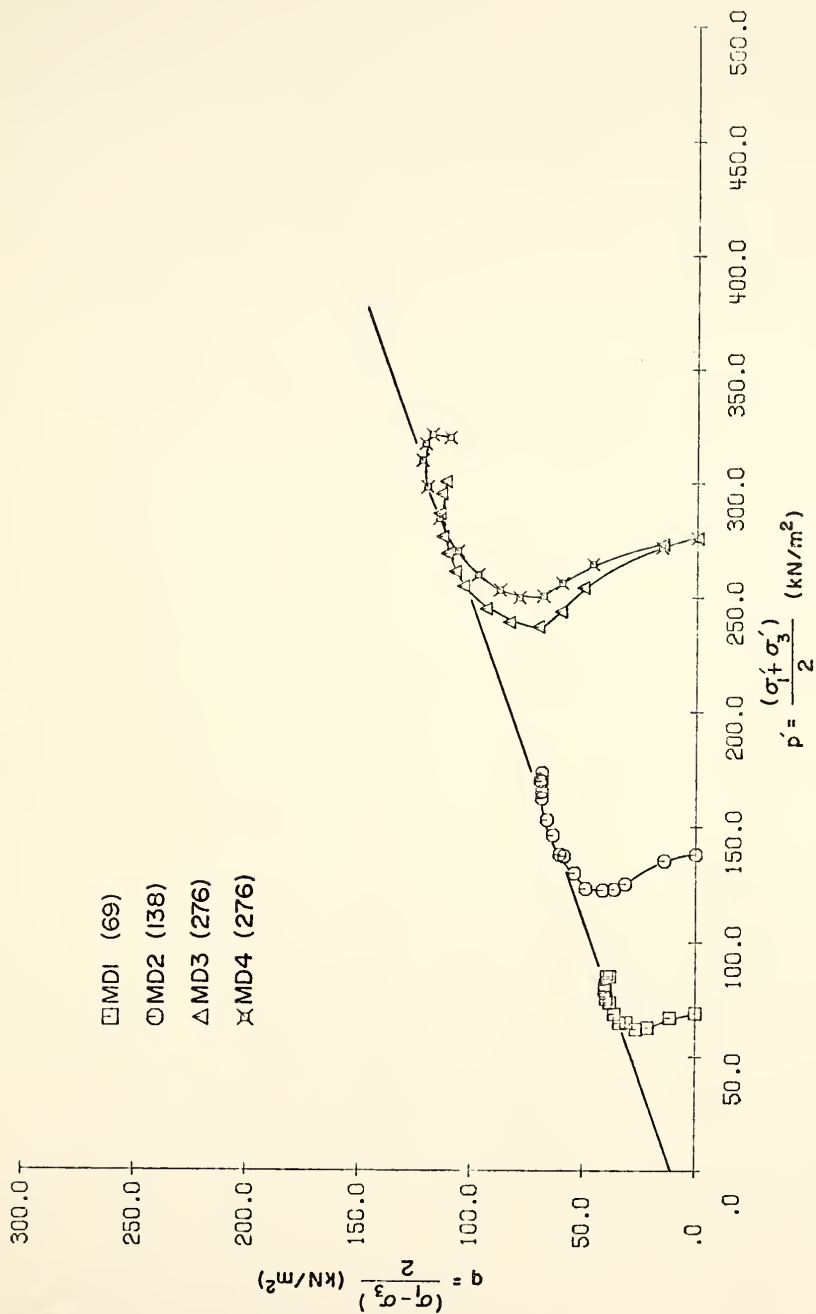


FIGURE D9 EFFECTIVE STRESS PATHS, TESTS MDI TO MD4

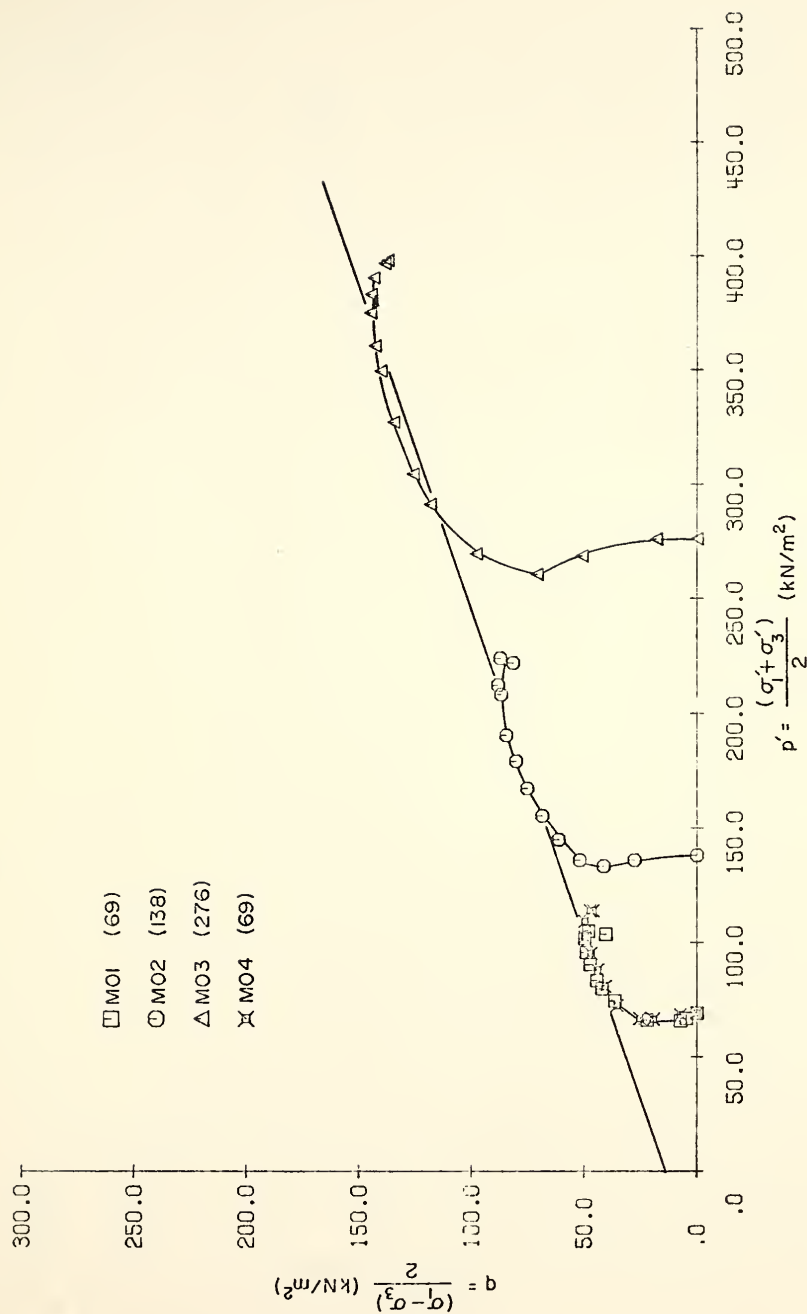


FIGURE D 10 EFFECTIVE STRESS PATHS, TESTS MO1 TO MO4

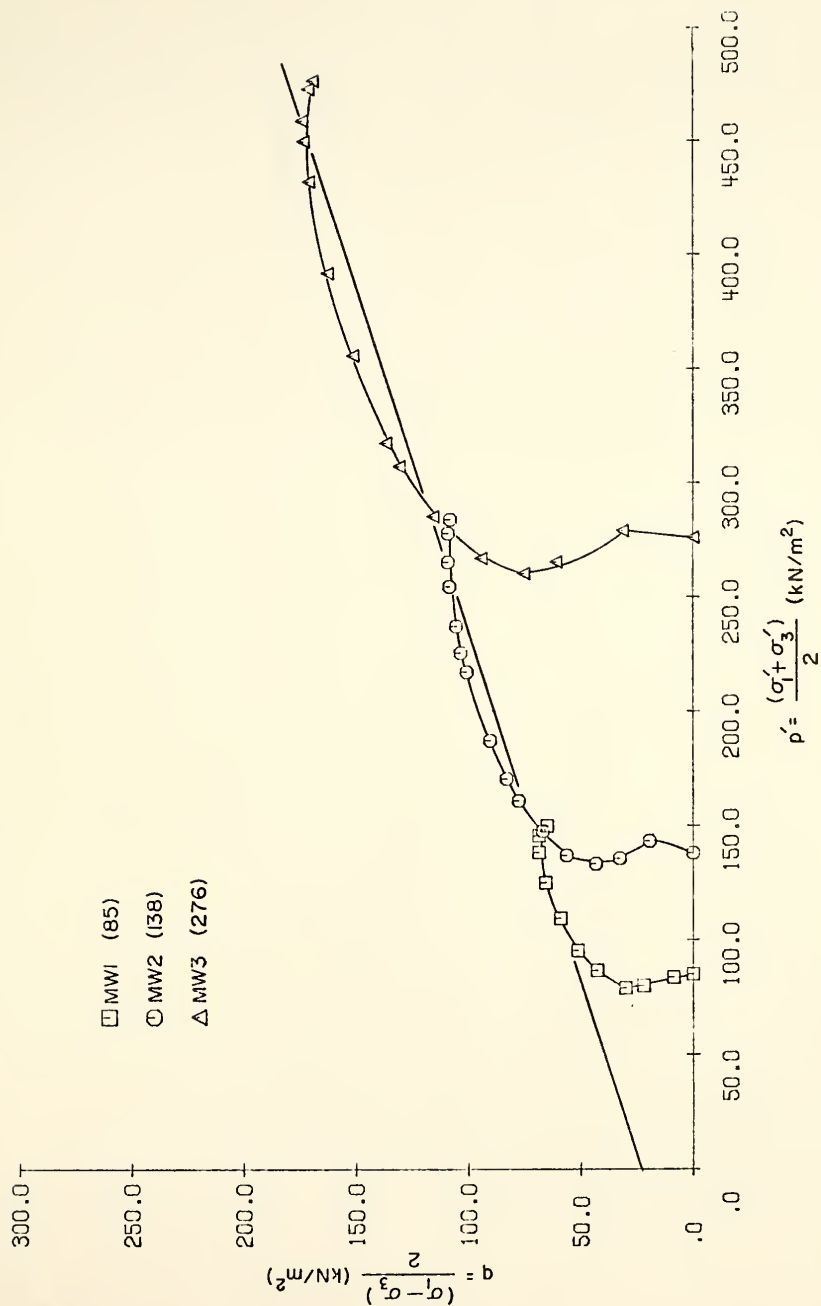


FIGURE D II EFFECTIVE STRESS PATHS, TESTS MW1 TO MW3

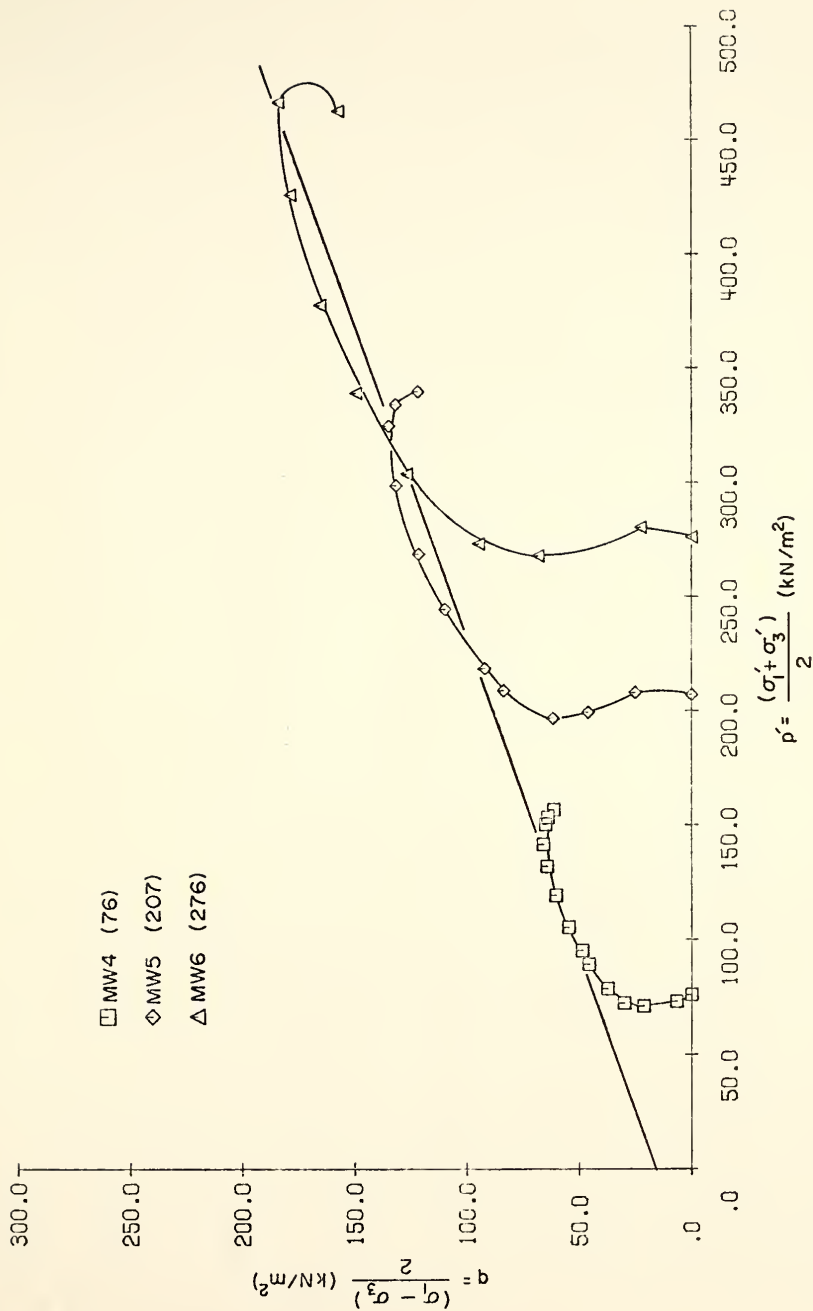


FIGURE D 12 EFFECTIVE STRESS PATHS, TESTS MW4 TO MW6

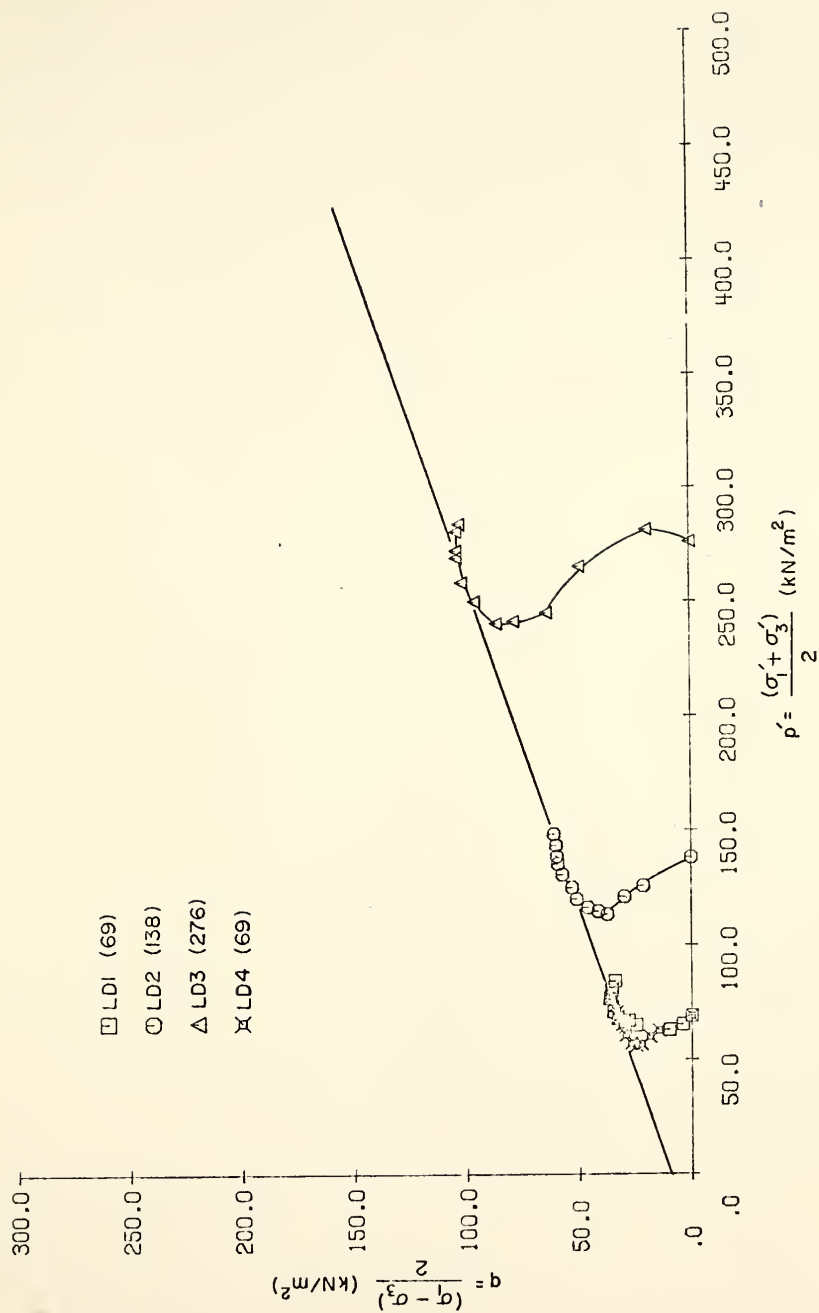


FIGURE D13 EFFECTIVE STRESS PATHS, TESTS LD1 TO LD4

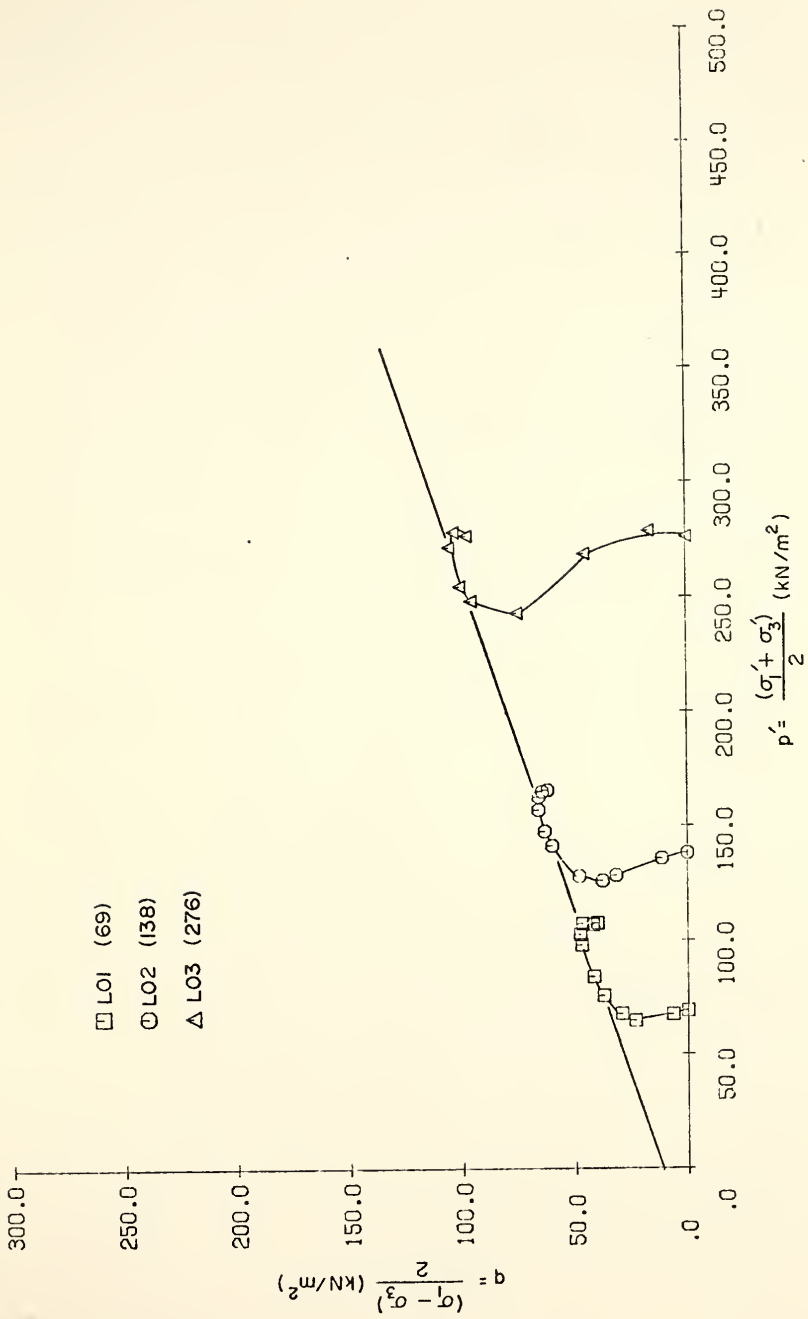


FIGURE D14 EFFECTIVE STRESS PATHS, TESTS LO1 TO LO3

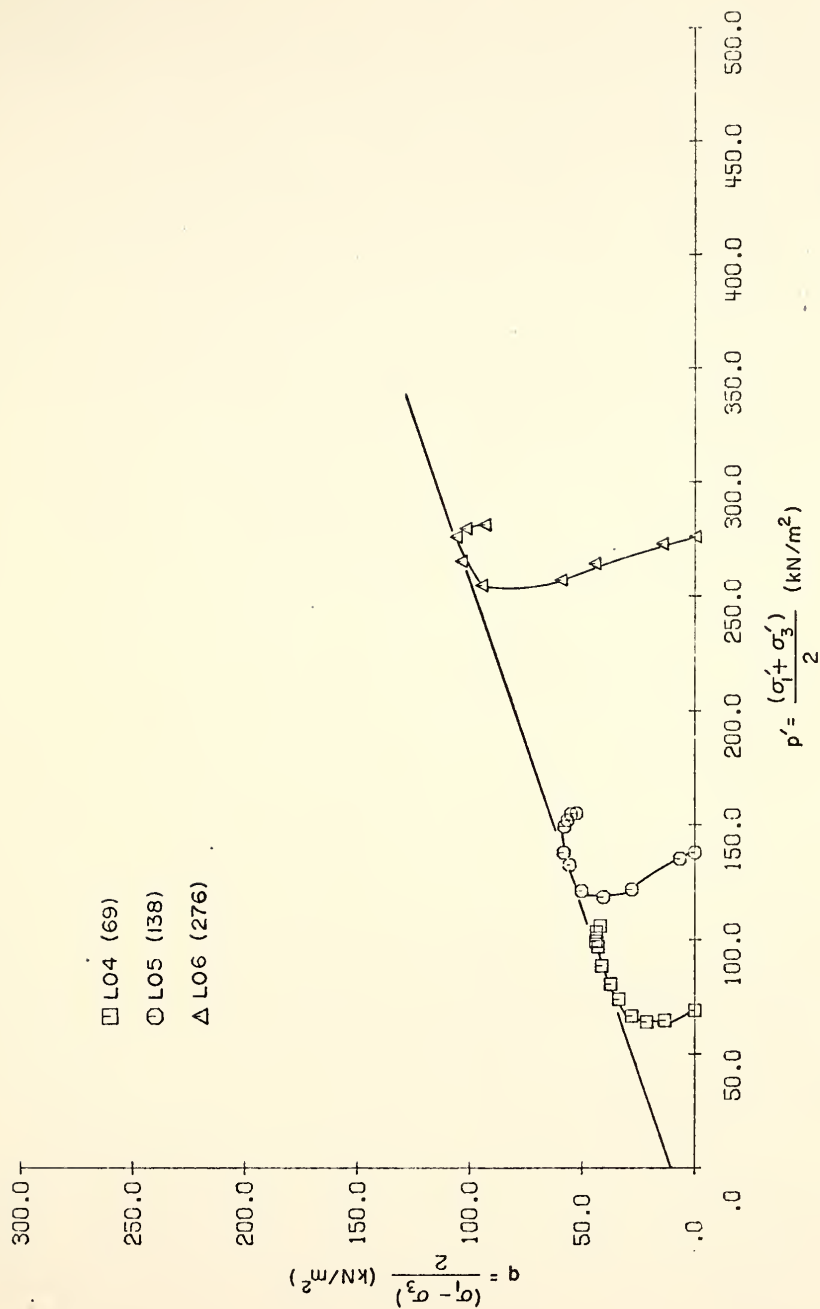


FIGURE D15 EFFECTIVE STRESS PATHS, TESTS L04 TO L06

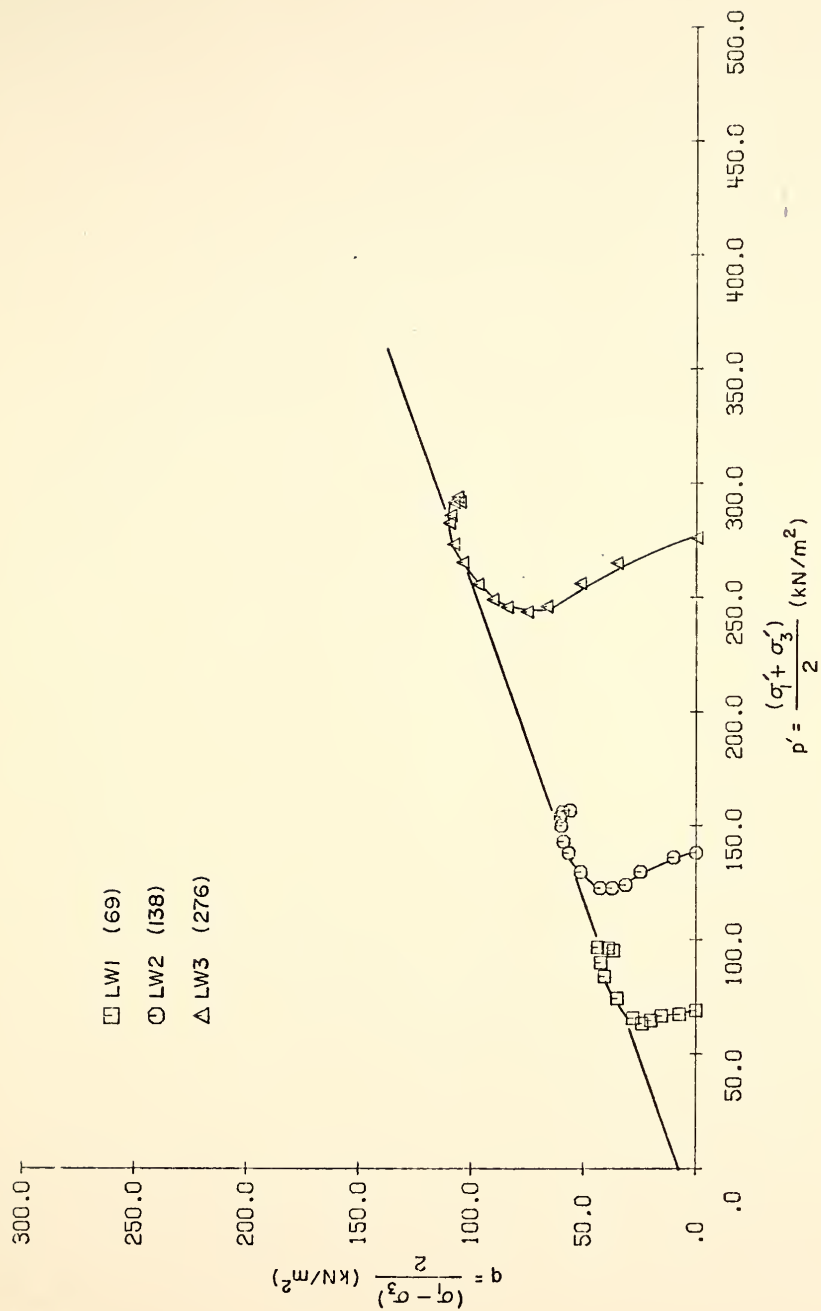


FIGURE D 16 EFFECTIVE STRESS PATHS, TESTS LW1 TO LW3

COVER DESIGN BY ALDO GIORGINI